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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

A Junior shall be not less than eighteen years nor more than twenty-five years of age, a student or connected with water supply work.

An Associate shall be either a person, firm or corporation engaged in manufacturing or furnishing materials or supplies for the construction or maintenance of water works.

A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

The initiation fees and annual dues are as follows:

	Initiation Fees		*Annual Dues*
Members	$ 5.00	Members	$ 6.00
Juniors	1.00	Juniors	3.00
Associates	10.00	Associates	20.00
Corporate Members	10.00	Corporate Members	10.00

This Association has at least eight regular meetings each year, of which five are held in Boston, one in northern New England, one in southern New England, and one, the annual convention, held in September on such date as the Executive Committee may designate.

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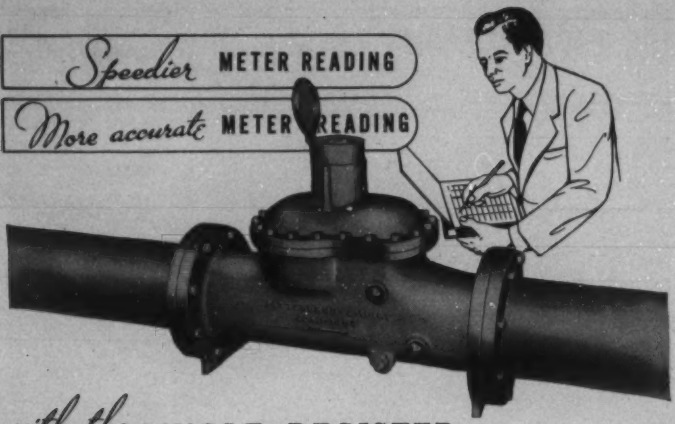

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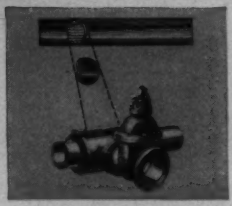

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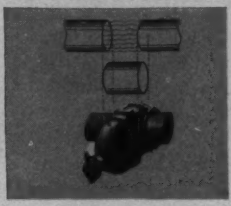

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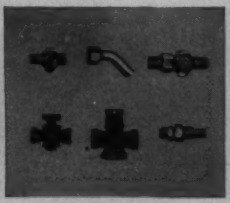

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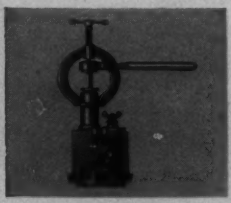

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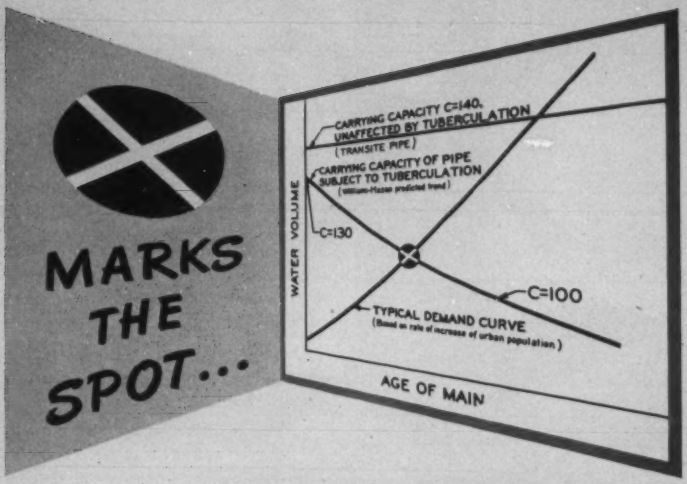
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AGE OF MAIN

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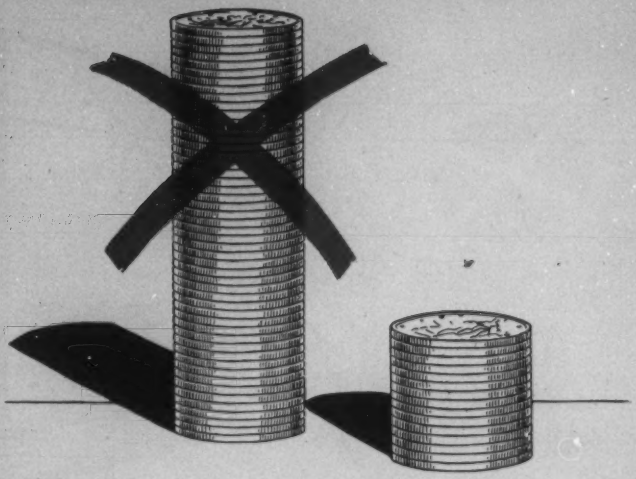

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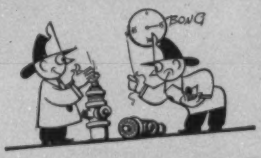

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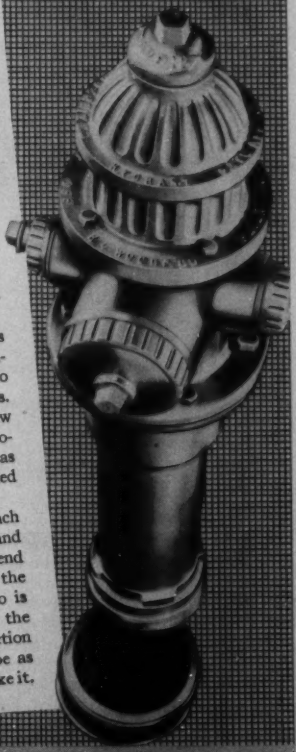

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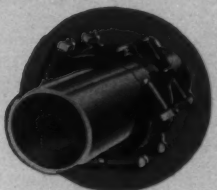
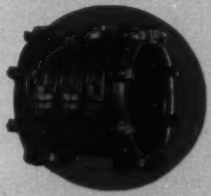


Style 82—Adjustable Repair Sleeve. *Low-cost, lightweight* sleeve to fit varying pipe diameters from 4" to 8". Interchangeable sections give adjustability. One man can make a repair with this sleeve in 5 to 10 minutes. Ideal as tapping sleeve.



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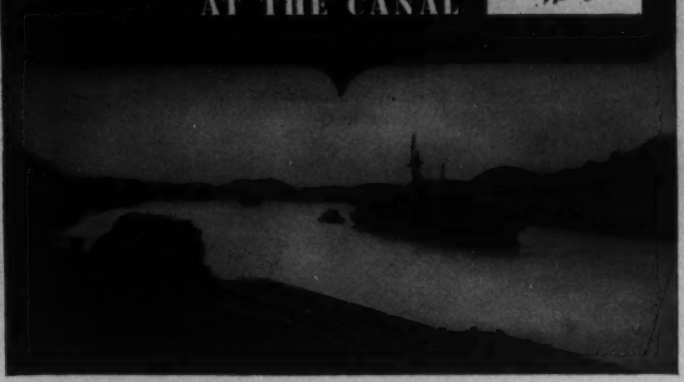
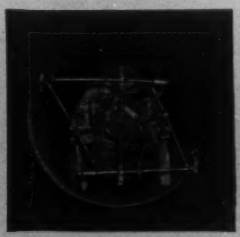


. . . **CENTRILINE** has recently reconditioned for the Government, the 58,000 feet of 30 and 36 inch cast-iron pipe line from Gamboa Pumping Station to Miraflores, Panama Canal Zone. The increased carrying capacity will now be permanently maintained by the thin, dense, smooth cement mortar lining centrifugally applied—a reliable guarantee against future tuberculation, underlying deterioration of metal and increased pumping costs.



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New England Water Works Association
ORGANIZED 1882

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This Association, as a body, is not responsible for the statements or opinions of any of its members

WATER-SUPPLY PROBLEMS IN LATIN AMERICA

BY CLARENCE I. STERLING, JR.*

[*Read September 11, 1945.*]

THE water-supply problems in Latin America are much the same as those facing North American cities, although they are usually more acute. This is due to the fact that the earning power of the people is less, as a result of which water rates must be lower, and to the fact that materials for construction and maintenance are more costly and often difficult to obtain. If you can remember how water-supply systems were operated in the United States twenty, thirty or forty years ago, you have some conception of the conditions in Latin America at the present time. I do not mean to say, however, that there are no up-to-date, modern water-supply systems in Latin America, for there are, but, with less income and higher material costs, the physical plants are generally inadequate as to capacity and the quality is often inferior to that found in the United States. These conditions are recognized by the competent engineers of those countries. There are so many problems facing each of the Latin American countries, however, especially in their economic development, that the cycle of refinement which we find in the United States in water-works practices and treatment has not been attained. Furthermore, the number of trained personnel available is small and there is little time or money for research and experimentation.

In the past, this has led the Latin American countries to follow the standards of European countries, especially as heretofore the technical training of Latin American engineers was obtained in Europe. In more recent years, however, the standards of the United States are beginning to be adopted.

Often we find that the water entering the distribution system is of good quality and treatment is sufficient to make it safe but there

*Chief Engineer, Division of Health and Sanitation, Institute of Inter-American Affairs, 499 Pennsylvania Ave., N. W., Washington 25, D. C.

are many hazards in the distribution system, such as cross-connections with sewers and secondary supplies, negative pressures, etc., which result in a higher incidence of water-borne diseases in the cities and towns having water-supply systems than you would expect to find. Distribution mains are usually of small capacity, with much lower pressures than are found in the United States, and the systems are not designed for fire protection as we know it. This is due to excess cost of installing the large mains that would be required to give proper fire protection and to the fact that most of the buildings are of masonry construction in the more important cities and towns.

The greatest problem in the existing systems is the lack of proper maintenance. This reflects the lack of trained personnel, political upheavals in management and insufficient revenue to finance repairs and extensions. Since relatively few systems have the service connections metered, there is usually a large wastage of water, principally in the private houses. For example, a recent pitometer survey made in San Jose, capital of Costa Rica, showed that 90% of the unaccounted-for water was wasted in the homes, either through leaky plumbing fixtures or by continuous flow of water from the fixtures. It is a common occurrence for sections of cities to be without water during periods of high consumption and there are even cases where water is pumped into the mains only at certain hours of the day.

When the Division of Health and Sanitation of the Institute of Inter-American Affairs started its coöperative health program in Latin America early in 1942, it was recognized that much of the work would have to be in the field of environmental sanitation. As you know, without good sanitation, it is impossible to develop an adequate public health program. Obviously, in any sanitation program the primary requisite is a safe and adequate water supply. Unfortunately, it was impossible to start many water-supply projects at that time, for waterworks materials were critical in the United States and shipping space was practically non-existent. In spite of these handicaps, water-supply projects of various types were carried out in many of the countries. The selection of the particular projects rested with the individual field party in the country but the selections which were made fitted into the general over-all policy of helping the war effort of the United States. Projects were developed for populations engaged in producing

strategic materials, in cities or towns supplying water to United States Army bases and airfields or to international commercial air fields, or in cases where it was necessary to improve the sanitary conditions of important cities and towns. To date approximately 63 projects, distributed as shown in the following table, have been planned or constructed by the engineers of the Division of Health and Sanitation.

PROJECTS FOR WATER SUPPLIES

	Engineering study only	Construction of water-supply systems*	Construction of water-treatment works	Minor water-supply projects	Total
Bolivia	2				2
Brazil		15			15
Chile					0
Colombia			1		1
Costa Rica	1	3	1		5
Dominican Republic					0
Ecuador	1				1
El Salvador	1	2	2	3	8
Guatemala		4			4
Haiti					0
Honduras		4	1		5
Mexico		13			13
Nicaragua	1	2	1		4
Panama					0
Paraguay	1				1
Peru		1			1
Uruguay				2	2
Venezuela				1	1
	7	44	6	6	63

*Some of these projects include treatment.

In addition, rural water-supply projects were inaugurated in many of the countries and small, safe water supplies were developed along the route of the Pan-American Highway in Central America.

Undoubtedly many of you are unfamiliar with the organization of the Institute of Inter-American Affairs. This Institute was incorporated as a Government corporation, a part of the original Office of the Coördinator of Inter-American Affairs. Several divisions are set up within this corporation, among them the Division of Health and Sanitation. When this Division was organized, the eminent authority on international health and tropical medicine, Major Gen-

eral George C. Dunham, was made Director. At the Conference of Foreign Ministers in Rio de Janeiro in 1942, one of the resolutions provided for the development of a coöperative health program in Latin America. Therefore, as each of the Latin American countries broke relations with the Axis, an agreement was made by the Institute of Inter-American Affairs, usually with the Minister of Health of the country, for the establishment of a coöperative health service and a sum of money was allocated by the Institute for financing this program. In all, coöperative programs were started in 18 of the American republics, the exceptions being Cuba and Argentina.

The basic agreement in each country provided for the establishment of a Coöperative Inter-American Health Service in the national health service of the country and a field party of North American technicians was sent to the country. These field parties consisted of a sanitary engineer, a public-health medical officer and a business manager. Either the medical officer or sanitary engineer was Chief of Party, depending upon the type of program which was expected to be developed in the country. Later other specialists were added, such as public health nurses, architects and entomologists, this personnel being added when and if required. The Chief of the Institute field party became the Director of the Coöperative Health Service and all of the other personnel in the Coöperative Health Service were nationals of the country in which they were working. The field parties determined their own program, following the general over-all policies laid down by the Institute. Therefore, with the delegation of authority to the field, it was necessary to obtain well qualified personnel.

At the present time we have about 65 persons employed in the Engineering Section of the Division of Health and Sanitation, 40 of whom are sanitary engineers. Before the war started, some of these engineers were associated with consulting engineering firms or state health departments and others were professors in sanitary engineering at some of our universities. Eighty per cent of the sanitary engineers are officers in the Sanitary Corps of the Army who were assigned to our organization, as it was recognized by the Army that the type of work we were doing was very essential to the war effort. Actually, the forming of the Coöperative Health Service was an entirely new

idea in the field of international relationships, for North Americans became a part of the national health service of a particular country and directed the functions and operations in coöperation with the national engineers and medical men.

It is interesting to note that many of the sanitary engineers were members of the New England Water Works Association or worked in New England before the war. You will be interested to know that our present engineering staff includes the following:

Major Wyman R. Stone, formerly with the Connecticut Health Department, now Chief of Party in Ecuador; Major Peter C. Karalekas, formerly with the Massachusetts Department of Public Health, now Chief Engineer in Honduras; Mr. Samuel T. Drew, formerly connected with Metcalf & Eddy, now Chief Engineer in Guatemala; Major John W. Greenleaf, formerly associated with Metcalf & Eddy and with Bayard F. Snow, now Chief Engineer in Paraguay, and Major Chester C. Ordon, formerly employed by Metcalf & Eddy and by the Massachusetts Department of Public Health, now a Sanitary Engineer in Ecuador. Captain James L. Dallas, who is on leave of absence from the Massachusetts Department of Public Health, was recently assigned to our organization by the Army after a tour of duty in India. He is being assigned to our Amazon River project in Brazil.

In addition, many of our engineers have had training in the Graduate School of Engineering at Harvard University and are undoubtedly known to many of you.

One of the questions that is asked us is, "What is the benefit of our work to the United States, especially as the war is now over?" There are a number of advantages to the United States in this cooperative program, for we believe that by improving the health of these people we will promote a better standard of living and ultimately a greater purchasing power. Before the war, much of the equipment and materials used in water-works systems was purchased in Europe. Now we are introducing equipment manufactured in this country and are creating a market for the North American water-works manufacturers. We also feel we are instilling American water-works practices into these countries, through close collaboration by our North American sanitary engineers, working in each of these countries, with

the national engineers. Mutual understanding and friendliness is being developed by the North American and Latin American engineers and both are obtaining mutual respect for their respective viewpoints. In this latter phase, a program has been developed by the Institute of Inter-American Affairs for bringing 25 to 30 eminent or promising engineers to the United States each year for graduate study in sanitary and public-health engineering. Many of you gentlemen have coöperated with us in giving these Latin American engineers practical field training, in addition to their academic training, and I wish to express the appreciation of our organization for your coöperation at a time when you have been so short of help and overworked.

You may also wonder if our work has been appreciated by the Latin Americans. While our files show many letters of appreciation from officials of the various countries, I truly feel that we have reason to believe that Latin American engineers have sincerely appreciated our work. This appreciation has been shown by the award of official decorations to some of our staff members by the Latin American governments, or by invitations to become members of some of the professional engineering societies. Further evidence of this appreciation is shown by the fact that, in the extensions of the basic agreements with the various countries, the Latin American countries are now contributing approximately $2.00 to our $1.00 for the coöperative health services. We also find that engineers in those countries are now eager to obtain fellowships for academic training in the United States and, with few exceptions, these men go home enthusiastic for the United States.

I feel that, if you men were fortunate enough to have worked with the Latin American engineers and people as I have, you would appreciate how important this work has been in preserving American solidarity during the war. Now with an era of peace before us, it is even more important that work of a similar type be continued to preserve that solidarity.

You will be interested to know that Colonel Harold B. Gotaas, Sn.C., has recently been made President of the Institute of Inter-American Affairs and Colonel John D. Yeagley, M.C., is Director of the Division of Health and Sanitation of the Institute.

ORGANIC COATINGS IN WATER-WORKS MAINTENANCE

BY D. R. MESERVE*

[*Read March 21, 1946.*]

A CASUAL glance at the field of synthetic resins discloses a bewildering array of new products, derived from an ever widening range of materials. Spurred and abetted by war demands, scientific researchers and technologists have steadily increased the number and variety of synthetic resins available to endow organic coatings with a multitude of desirable properties. These very advancements, however, create serious perplexities when it comes to choosing the right resin for a particular type of protective coating or finish. This is due to the fact that not only do different synthetic resins behave differently under similar conditions, but resins can be compounded one with another to exhibit characteristics which they do not possess singly. However, by understanding the basic nature of synthetic resins, as indicated by the materials from which they are derived, and how they function, the user of organic coatings stands a better chance of obtaining the right formulation for his particular conditions.

Studious examination of not merely a few synthetic resins, but of all that show any likelihood of promise for use in a corrosion-resistant coating, has steadily added to the fund of scientific data that can be placed at the disposal of the user of organic coatings.

As a first step in establishing the individual advantages offered by various synthetic resins, it might be well to understand how they work in general. Usually but not always, resins are in solid form. To function as organic finishes, they are dissolved in a solvent or blend of solvents, with certain modifying resins and plasticizers added to give desired results. In order to assure uniformity of the finished material, such factors as temperature, speed of agitation and methods of dissolving must be accurately controlled.

When the solid resin is treated with the proper solvent, it actually absorbs the solvent. The resin swells, a gel structure develops,

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the plasticity increases but the mechanical strength decreases. On the other hand, once the correctly formulated organic coating is applied as a protective finish, it is just the other way around. A gel is formed, plasticity decreases and mechanical properties increase, until a firm film results. Coatings reach their final stage in three different ways, depending upon the nature of the resins employed. These ways are evaporation, as with lacquers; oxidation, as with oil paints and varnishes, and polymerization, as with synthetics. Furthermore, one or more of these changes may take place at the same time with the same coating.

Important factors in the "blue-printing" of the constituents of any coating are the conditions to be encountered in service. Other important considerations, however, must be dealt with before a material can be recommended. For instance, a finish based upon a certain synthetic resin may have a desirable set of properties but it would prove difficult to work under certain conditions. This difficulty in handling can impose an operational or cost burden which may defeat the advantages offered by the resin. The problem is, then, to "engineer" an organic coating that will produce the desired finish and be economical and easy to use.

Some individual resins have natures and properties which are highly desirable in formulating organic coatings. Often, however, by combining different resins, even more desirable characteristics can be developed. The variety of such combinations is almost limitless, in formulating organic coatings to do specific jobs and to meet the conditions imposed by a myriad of end-uses.

It seems advisable at this point to describe some of the resins now used in protective coatings.

Nitrocellulose, the first member of the cellulose family, is produced by treating cotton linters with mixtures of nitric and sulphuric acids. It has long been used in protective coatings, because it has the advantage over old varnishes of drying extremely fast. In earlier days, it had certain disadvantages of viscosity and adhesion. Newer techniques in its manufacture and the incorporation of other resins, plasticizers and solvents have eliminated many of the former difficulties.

Today nitrocellulose resins are used in producing organic finishes

with an excellent surface, that is tough and long-lasting, with good resistance to many dilute acids and alkalies. Generally, finishes formulated with nitrocellulose resins have good resistance to gasoline and water and fair resistance to mineral and animal oils, but are attacked by salt water, alcohols and vegetable oils. An objectionable characteristic is that nitrocellulose is highly inflammable. In addition, it will discolor upon prolonged exposure to sunlight and is inclined to become hard and brittle with aging.

Cellulose Aceto-esters. Here is another member of the cellulose family but this one is derived from treating wood or vegetable fibres with acetic acid and acetic anhydride, instead of sulphuric and nitric acids. The properties of these resins depend upon the degree of acetylation, several grades being commercially available.

As for the properties of cellulose aceto-esters, these resins are light-resistant, slow-burning, resistant to salt spray and gasoline, but do not have good resistance to acid and alkalies. Their toughness makes it very difficult to obtain the proper degree of adhesion, so that their use in coatings for metal is limited.

Ethyl Cellulose. This member of the resin family is manufactured from cellulose—derived from cotton linters or wood pulp—caustic soda and ethyl chloride. In it we find still other properties.

Ethyl-cellulose resin provides a softer film than those derived from the treatment of wood or vegetable fibres with nitrates or acetates. This resin resists attack by hot, concentrated alkalies but does not have good resistance to acids, moisture, gasoline and oil. In its favor is the fact that it will stand exposure to sunlight without discoloring—plus good electrical properties, toughness, and high-and-low-temperature flexibility. Its chief use in solution coatings is as a modifying agent in varnishes and nitrocellulose lacquers.

Vinyl Resins. This is another family in the world of synthetics. Its primary source is petroleum or coal. There are various types of vinyl resins having widely different characteristics. Of most interest in the surface-coating field are the copolymer vinyl resins, several grades of which are produced. These resins vary both in degree of polymerization and in the proportion of vinyl compounds. One of their outstanding properties is extreme inertness at ordinary temperatures. They have excellent resistance to practically all acids, alkalies

and oxidizing agents and are not affected by gasoline, greases, alcohol or water. They are colorless, tasteless, odorless and non-toxic. Among the most versatile of all synthetic resins, vinyl resins are used in organic finishes having a broad field of application—from coating beverage and food cans with an odorless, tasteless, non-toxic film to coating tanks and tank cars where chemical resistance is the essential requisite. They are widely used for protecting alkaline surfaces, such as concrete, and for coating industrial equipment as a protection against chemical fumes.

Acrylic Resins. Although these resins are also derived from petroleum or coal, their properties differ widely from those of the vinyl family to which they are related. Some are hard, brittle solids, while others are soft plastics. Acrylic resins are characterized by their water-white color, resistance to discoloration and perfect transparency. They have good resistance to water, alcohol, acids and alkalies. However, it is difficult, when using these resins as a primary base, to formulate a coating having good adhesion and good flexibility with adequate hardness.

Their chief value is as modifying agents, to improve water-whiteness and resistance to discoloration of lacquers having other resins as a primary base.

Polystyrene Resins. Like the vinyl and acrylic resins, the polystyrene resins are derived from petroleum or coal. These resins have excellent resistance to acids, alkalies and water, but poor resistance to gasoline, oils and greases. At the present time their use in organic coatings is limited, due to difficulty of obtaining satisfactory adhesion and flexibility. However, these difficulties may be overcome by the development of new plasticizers and modifying resins.

Vinylidene Resins. The primary source of these resins is petroleum or coal and salt. They have excellent resistance to acids, alkalies, water, gasoline, mineral and animal oils, and greases. An outstanding characteristic is their high impermeability to moisture vapor. Their main disadvantages as a coating for metal surfaces are their inherent lack of adhesion and a certain chemical instability at elevated temperatures.

Alkyd Resins. There are probably more different modifications and components among the alkyd resins than in any other resin

family. These resins are derived from coal or grain and vegetable oils. Modifying agents are so wide in range that it is almost impossible to list them all. Their advantage is that, with alkyd resins, the plasticizing agent becomes an integral part of the resin molecule and provides internal plasticization. In addition to insuring a high-gloss film, alkyd resins in general offer good adhesion plus excellent weather resistance and exceptionally long life. Resistance to water, grease and chemicals, however, is not as good as that offered by the vinyl-type resins.

Urea Resins. Urea resins are evolved from coal. In combination with other resins they form coatings which are water-white, do not discolor and are especially resistant to marring. They have good resistance to oils, greases and waxes, and fair resistance to weak acids and alkalies. In view of the fact that they are thermosetting, they must be cured at elevated temperatures. In combination with alkyds, they have excellent adherence to metal surfaces and are especially desirable for finishes on metal products.

Phenolic Resins. Here is another resin family, stemming from a distillate of hard wood or from petroleum, which offers an endless number of combinations. Desirable properties may be enhanced and undesirable ones modified by the addition of a large number of oils and resins. Phenolic resins can be divided into two main classes—thermoplastic and thermosetting. This latter category must be subdivided into spirit-soluble resins and oil-soluble resins. The spirit-soluble resins yield hard, glossy, abrasion-resistant films. Such films offer good resistance to solvents, acids and chemicals, but resistance to strong alkalies is poor. Oils, chemical plasticizers and most other resins are not compatible with them. Except for exhibiting brittleness under extreme flexing or other rough mechanical treatment, the resulting finish offers an unusual combination of useful properties.

The oil-soluble thermosetting resins are soluble in such vehicles as tung or linseed oil. Coatings with these resins are generally used as baking finishes. They cure well in thick films and insure a coating having excellent electrical properties.

Of all the 100%-phenolic types, the oil-soluble thermoplastic resins are used most. An outstanding characteristic of a coating formulated of these resins is its resistance to outdoor exposure.

Melamine Resins. These resins, compounded from coal and lime, are also thermosetting. They produce coatings which have better water-whiteness than ureas and are more resistant to discoloration. They can also be cured faster or at lower temperatures than the ureas. An outstanding characteristic of melamine coatings is their excellent outdoor-exposure resistance. They are used where exceptional color retention, combined with resistance to abrasion and outdoor exposure, is a prime requisite.

From the foregoing "refresher course" on the time-honored resins as well as the newer ones, it is apparent that: (1) many synthetic resins have desirable properties in common; (2) many have individual characteristics that recommend them for a particular purpose, and (3) by a skillful compounding of resins, plasticizers, solvents and inhibitors, still other advantageous properties can often be developed.

It is this studious job of analyzing the capabilities of various synthetic resins and ascertaining how to bring out their best qualities, which has aided the development of organic coatings offering a maximum of desirable properties for a specific service. These coatings are the end-products of carefully planned research activity—constantly on the trail of new and better materials and methods of manufacture and use.

During our investigation of these resins we have had one particular thought in mind—that is, to develop a coating that would be resistant to extremely corrosive conditions. Certain of the resins which we have discussed seem to have a wide range of resistant properties, as well as many other attributes which lend themselves to this type of application.

At the present time there are 10,656 water works and 7,488 sewage plants in operation in the United States. Thirteen states are now considering water-works projects estimated to cost more than $10,000,000 each and seventeen states are considering sewage projects estimated to cost more than $10,000,000 each. All of these plants and projects have extremely corrosive conditions with which to contend.

Let us look at specific corrosion conditions that are encountered in plants of this type. In the case of water works, we immediately

think of miles of piping. Where distribution-system piping passes through acid and alkaline soil conditions, it is a tremendous problem. In 1922, the National Bureau of Standards began a series of tests to determine the corrosion action of various soils toward the commonly used ferrous pipe materials. In their work, alloys, metallic coatings, bituminous coatings and various organic coatings have been tested. Since these tests run for 2 to 17 years before completion, test data for many of the newer synthetic resin coatings are incomplete, while other coatings have not been tested at all.

For example, phenolic resin coatings have been applied and tested. The test data on these coatings give them a perfect record after two years' exposure to all types of soil conditions. A film formed by a phenolic coating is characterized by durability under conditions of exposure, in which resistance to high humidity, water and strong chemical reagents is necessary. The film is smooth, glossy, non-porous, hard and flexible enough to withstand distortion, when used as linings in railway tank-cars now in service. It is obvious that this type of coating would find many applications in the metal-pipe lining and coating field. The main disadvantage encountered with a material of this nature is that best resistance characteristics are obtained only when the material is baked. In general, pipes coated with phenolic resin finishes are received from the manufacturer already coated.

In water-works tank houses, which house chemical tanks and equipment, the protection of interior and exterior surfaces is advisable. In this instance, the metal work, pipe and concrete are subject to chlorine fumes and moist air, as well as other types of chemicals used in dosing. Corrosion-resistant coatings will add many years to the life of units of this type.

For example, let us take the case of a large Midwestern filtration plant, which is reputed to be a fine water-works project. At this plant water is pumped by a low-service pumping station from a reservoir of 210-mil gal capacity, formed by a dam in a river. It is brought into the filtration plant through an aeration basin, where the water is sprayed into the air to oxidize and drive off organic vapors. The basins are concrete with some metal equipment and framework. They must be protected against erosion from the swirling river water, carry-

ing abrasive particles, as well as against corrosion from the water and the various chemicals that it carries. The next step is to the primary mixing tanks, where lime, alum and occasionally activated carbon and soda ash are added to and mixed with the river's water. Here again the tanks are concrete with metal equipment. These are protected with organic coatings. From here the water is taken into coagulation tanks, where the mix is repeated and floc formation is promoted. The constantly swirling water and chemicals in these tanks again indicate the use of a suitable organic coating. There is a place for a corrosion-resistant coating in each succeeding step through this plant: primary clarifier, primary carbonation—where the carbon dioxide will corrode away concrete quite rapidly—secondary mixing tanks—where chemicals are again added—secondary clarifier for sludge removal, settling tanks, secondary carbonation and filters, directly followed by addition of gaseous chlorine, prior to storage in a 20-mil gal reservoir.

The high-service pumps from this reservoir have the capacity to deliver 60 mgd. With all of this water passing through concrete and metal pipe, prior to and during treatment, the corrosion problem involved is an extensive one.

Many filtration plants have their own units for the manufacture and purification of carbon dioxide for their carbonation processes. This carbon dioxide is generated and then forced through large cylindrical tanks, known as "scrubbers." Here the carbon dioxide is sprayed with water, as it passes through the chambers. In most cases, these steel cylinders are lined with bituminous mastic and occasionally with concrete grout. Their life expectancy in these cases is about 9 years. It has been recently shown that, by using a three-inch grout lining, coated with multiple coats of an air-drying protective coating, the life expectancy can be greatly increased.

In all of the applications that we have mentioned, phenolic resins, vinyl resins and vinylidene resins are especially adapted to the conditions outlined, due to their inherent chemical and moisture resistance. Resistance to chlorine is a prime factor and vinyl-base coatings will stand aqueous solutions with as much as 15% available chlorine—well over any concentrations used in water-works systems.

In sewage disposal plants, protective coatings are again used

on pipes, equipment houses for screens and racks, concrete floors, tanks and walls, chemical- and gas-storage tanks, dosing equipment, digesting equipment, de-watering equipment and sedimentation-tank mechanical equipment. In general, the requirements in this type of plant are resistance to water, chlorine and sewage, which contains organic acids.

The never-ending battle against corrosion requires detailed attention to each specific type of attack. The versatility of the resins that have been developed in the last few years has led to the development of correspondingly versatile surface coatings. Since the universal protective coating which can be applied under all conditions for all purposes is yet to be discovered, the surfacing engineer must consider all phases of a given problem before making his recommendation. A vast majority of coating problems is solved by these men each day and, as our fund of knowledge increases—who knows?—perhaps the universal coating will still be discovered!

ELECTRONIC PRINCIPLE OF WATER PURIFICATION

BY FRED E. STUART*

[*Read April 18, 1946.*]

THE use of a coagulant for precipitation or floc formation of impurities to be removed is well known and needs only a passing reference. The most common coagulant used in this connection is aluminum sulphate, also called filter alum, which is added to the water in predetermined dosages by dry-feed equipment ordinarily in order to ionize the water. The application of filter alum, having acid characteristics, reduces the pH and therefore requires further careful proportioning of lime, which has an alkaline characteristic, for the proper adjustment of the pH value.

The electronic purification principle provides a method by which alkaline or acid solutions, containing concentrations of color-bearing materials, may be coagulated under conditions requiring materially less control than the present application of filter alum and hydrated lime for the resulting formation of hydrous aluminum floc for color removal.

This improved method of purification utilizes the mass action of the surface area of sheet aluminum and, by dividing the liquid to be treated into slices of solution, increases the velocity of coagulation and produces instantaneous floc formation, without the employment of the usual filter alum. The electronic purification method, in which the use of filter alum is discarded, increases the velocity of coagulation by the employment of electricity.

Figure 1 shows a simple detail of construction of the electronic purification unit, termed "flash coagulator" by some investigators, which is quite simple in construction and allows an easy replacement of parts, for the aluminum electrode units must be replaced as the aluminum dissolves with use. This illustration is for the smaller installations, where small rates of flow form hydrous aluminum-floc slurry, which can be used for decolorization of chemical solutions and

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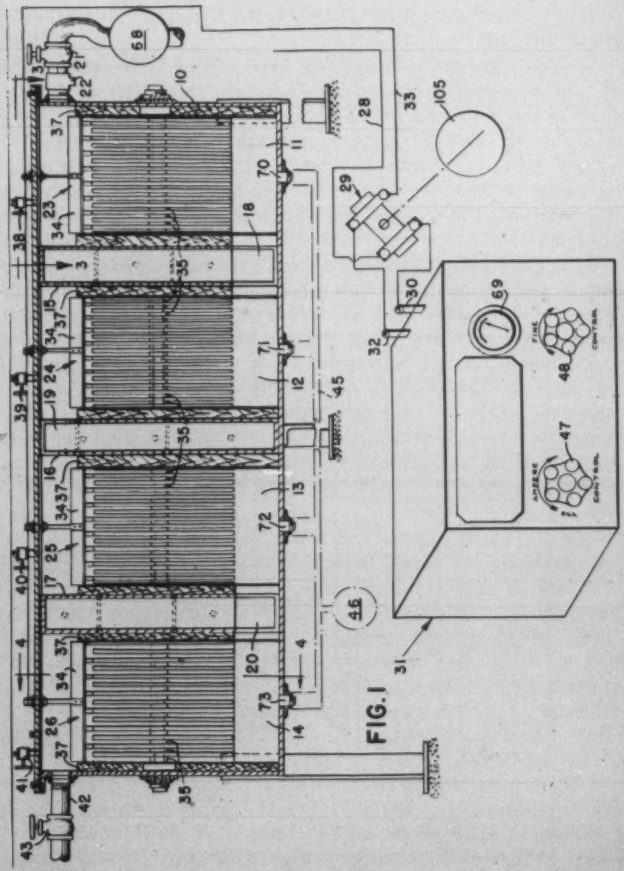


FIG. 1

other liquids to be reduced in color by the slurry produced in the electronic coagulator, rather than passing the water to be coagulated through the unit.

Figure 7 shows an elevational view of the flash-coagulator units arranged for installation in flumes, where the water to be treated passes through the baffled areas of the mixing chamber. Figure 8 shows the top view of the arrangement as shown in Figure 7. In handling water of extremely difficult characteristics, such as industrial wastes, it has been found advisable to revolve the electrode units in the solution to be treated. Such an arrangement is shown in Figures 11, 12 and 13.

General Outline. Instead of using the ion in filter alum, as is common practice in water works for color removal, according to the electronic coagulation principle, the aluminum ion is removed from aluminum sheets by the use of direct-current electricity of low voltage and low amperage, the voltage and the amperage varying according to the mineral content of the water. The water to be purified is passed through the aluminum electrode units connected to a source of direct-current electricity (see Figure 1). This causes the practically instantaneous formation of a tough floc, which is not easily broken up and can be handled much more roughly than ordinary alum floc and which can be removed by settling and filtration, as outlined above. Instead of using the ion in filter alum, the same type of floc formation can be produced by the electrode units of Figure 7, where the electrodes are alternately moved up and down in the water to be treated. This arrangement is designed for larger installations, where higher rates of flow are required for treatment. With this arrangement the electrodes are kept clean and more effective for floc formation.

Operation. With particular reference to Figures 1-4, the arrangement shows that the water to be treated is pumped by a pump 68 through valve 21 and strainer 22 into the upper portion of compartment 11. The switch 29 occupies the position shown in Figure 1 and conveys direct current to various electrode units. The water flows downward in compartment 11 through unit 23 and from there through the opening 18 into compartment 12. The water then flows upward through the unit 24 and the opening 19 into compartment 13, where it passes downward through the unit. From here the water passes through the

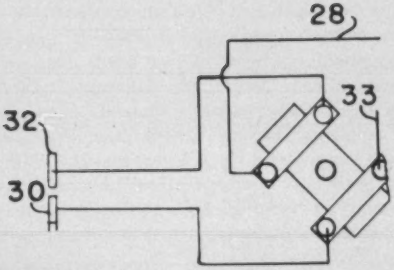


FIG. 2

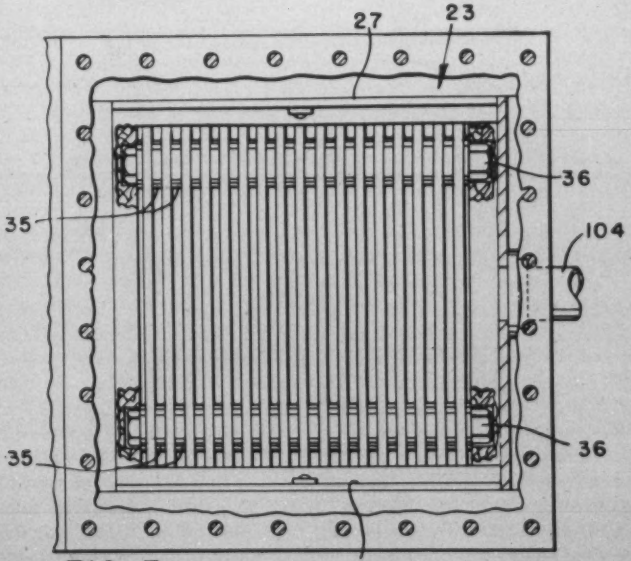


FIG. 3

opening 20, through compartment 14 and upward through the unit 26. While this flow of water occurs, the aluminum ions are split off from the negative aluminum electrodes to bring about coagulation. The water appearing in the upper portion of compartment 14 will be smoky and this smokiness can readily be seen to be an aluminum hydrous oxide floc formation. As soon as this point is reached the ammeter 69 may read 8 amperes. By turning the fine control 48 until the ammeter drops to 5 or 6 amperes and the smoky characteristics of the water coming through the affluent valve 43 are suitable, any variation can be obtained in the type of floc formation by regulating the valve 21 and the fine control 48.

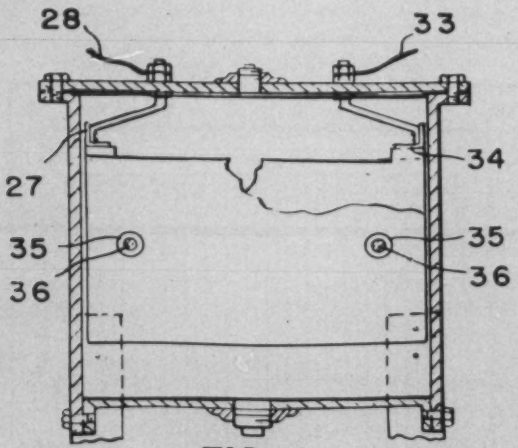


FIG. 4

During the operation of the electronic coagulator the water is cut into slices of solution, because it has to flow between the control spaces of the aluminum sheets of the various electrical purifying units 23, 24, 25 and 26. By cutting the water into slices of solution, the electrical energy from the plate is in continuous and controlled contact with the entire body of water passing through the units, as all

of the aluminum plates contain the same amount of current and all of the water must pass through the spaces between the plates.

The mineral content of the water unites with some of the aluminum ions from the aluminum plates and, due to the electrical activity of the watery solution, an aluminum compound is formed, which instantaneously removes organic color material from the water. The aluminum compounds can be flocculated out of the water by this electrical treatment without lowering the pH value, as is the case when applying filter alum and other acid-bearing chemical coagulants, which depend on the alkalinity of the water for floc formation.

If any gases should develop as a result of over-treatment in the container 10, these gases may be released through valves 38, 39, 40 and 41. These valves may also be used for releasing foamy floc to waste or for the application of a split treatment, when the electronic coagulator is allowed to generate excess aluminum hydrous oxide compounds for split treatment.

In order to control completely the action of mineral salts in the water and electricity, it is necessary at times to bubble air through the various compartments or to use high-velocity wash water. This can be effected by means of the source of air pressure or high-velocity wash-water pressure 46 and the pipe system 45, as well as the opening of the closures 70, 71, 72 and 73 in the bottom of the container 10.

It will be noted that only one group of the aluminum plates in each electrode unit breaks down at a time—namely, that group from which the aluminum ions are split off. Therefore, in order to obtain even wear of the various electrodes in each unit, it is necessary to reverse the polarity of the electrodes. This is easily effected by turning the switch 29 from the position shown in Figure 1 to the position shown in Figure 2. Preferably a timer 105 is provided, which automatically, after a predetermined time, changes the position of the switch 29 from that in Figure 1 to the position of Figure 2. The smoky aluminum-compound floc is now passed from pipe 42 into the pipe 50, where it passes through the openings 52. The floc may be directly withdrawn through valves 53, 54, 55 and 56 into the waste pipe 57, when excess treatment is applied, as this type of floc will float.

With routine treatment, the water can be flocculated, so that the

floc will settle in ordinary settling tanks, and filtered through sand or "anthrafilt" filtering material, whence it passes on into the reservoir.

Various tests of the electronic principle have been made in different sections of the United States: namely, the Dismal Swamp area of eastern North Carolina, soft surface supply, pH 5, color 400; Okeechobee Swamp water in Florida, hard surface supply, pH 7.5, color 120; Lake Erie water, city of Cleveland, raw water, medium hard, pH 7.5, alkalinity 75; Lake St. Clair water, city of Detroit, raw water, pH 7.9, alkalinity 83. After the above tests, among others, it can be said that the electronic principle of coagulation yields a higher degree of purification and a faster floc formation, commonly termed "flash coagulation", than have heretofore been obtainable with routine chemical coagulants. The electronic coagulator does not require the same size of settling basins as do the regular chemical mixing units of present-day practice, as the electronic flocs are much tougher and can actually be passed through flash mixers without tearing up and going back into the solution. Flocs settle fast and require less careful treatment. Therefore smaller settling basins can be utilized.

Another very important advantage of this type of coagulation consists in the fact that low-alkaline high-colored water can be coagulated for color removal, while the pH value can be increased at the same time—which is the reverse of alum coagulation. A typical example of this is the treatment of the Dismal Swamp water of eastern North Carolina, with a color of more than 400 and a pH value of less than 5, which was successfully treated to a color reduction of 10 and a pH value of 7.6 in thirty seconds' contact time in the electronic coagulator. In addition thereto and most important, hydrated lime could then be added to this electronically coagulated water, to bring the pH value up to 9.2 without an increase in color, which cannot be accomplished with present-day treatment by alum and lime and other acid coagulants.

A further feature of the flexibility of the electronic principle consists in the fact that the pH value of a lime-softened water can be reduced from 10.1 to 8.6 by simply passing the water through the electronic unit for thirty seconds, according to the principles described above. A specific example of this is the city of Miami's softening-plant experiments, which were conducted by the writer.

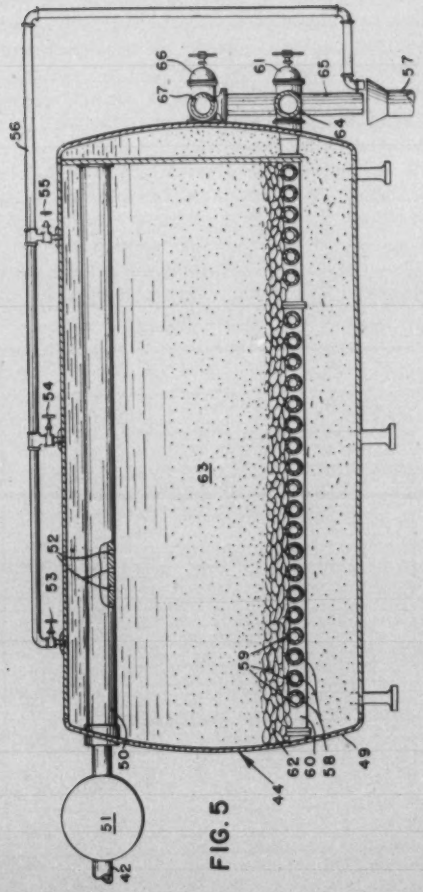


FIG. 5

The electronic method is also applicable to the removal of true chlorine residuals of low concentrations. For this particular application, it is advantageous to arrange the electrode units shown in Figure 1 in such a manner that they will not obstruct the flow of water. More specifically, instead of using the arrangement shown on a larger scale in Figure 3, the unit may be turned 90°. In other words, on the assumption that the water enters through the pipe 104 shown in Figure 3, it is clear that the flow of the water will be obstructed to a certain extent by the electrodes, whereas, if the electrodes are in the position shown in Figure 6, the water entering the unit through pipe 104 will not be obstructed by electrodes across the flow. Since they will be in line with the flow, they will reduce the loss of head through the unit and allow a much higher rate of flow.

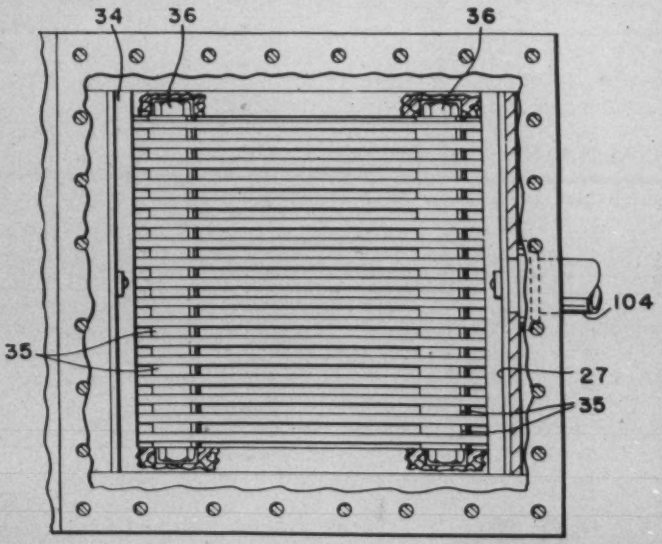


FIG. 6

It has been found with the electronic coagulation principle that the chlorine-demand reduction is almost directly proportional to the percentage of color removal.

Structural Arrangement of Stationary Unit. As shown in Figures 1-4, the stationary electronic-cell arrangement comprises a container, designated as 10, which is subdivided into compartments 11, 12, 13 and 14 by partitions 15, 16 and 17. The partitions are so arranged that partition 15 has an opening 18 at the lower end, while partition 16 has an opening 19 at the upper end and partition 17 has an opening 20 at the lower end. In this way, the water passing through the various compartments is forced to flow over and under through the entire compartments. The water is admitted through gate valve 21 and strainer 22 into the compartment 11. The strainer will retain any coarse particles or elements which may damage the electrodes. Inside each compartment is mounted an electrode unit, designated 23, 24, 25 and 26, consisting of closely spaced aluminum sheets or electrodes, through which direct current is passed, while the water is allowed to flow between these aluminum plates or sheets at varying rates of flow for the production of aluminum-bearing floc compounds, which remove color from the water. It has been proven advantageous to space the aluminum plates $\frac{1}{8}$ in. from each other. As will be seen from Figures 4 and 1, every second aluminum electrode is connected by an angle 27, which in turn is connected to a wire 28, leading through a switch 29 to one contact 30 of a rectifier unit 31. The other contact 32 of this unit is connected through switch 29 with a wire 33, leading to an angle 34 connecting the remaining electrodes of the same electrode unit. The individual electrodes are spaced from each other by plexi-glass spacers 35 (see Figure 3). While all electrodes are held together by bolts 36, the electrode units are mounted on wooden walls, connected to the container 10 and the partitions 15, 16 and 17, respectively. The inner sections of the containers are also lined with plexi-glass between the electrodes and the wooden walls. Valves 38, 39, 40 and 41 in each of the compartments 11, 12, 13 and 14 allow the removal of floc. Compartment 14 is connected to pipe 42 with a valve 43, the pipe 42 leading to a settling basin or filter 44 (see Figure 5). Each of the compartments is provided at the bottom with a pipe system 45, connected with a source of compressed air or high-

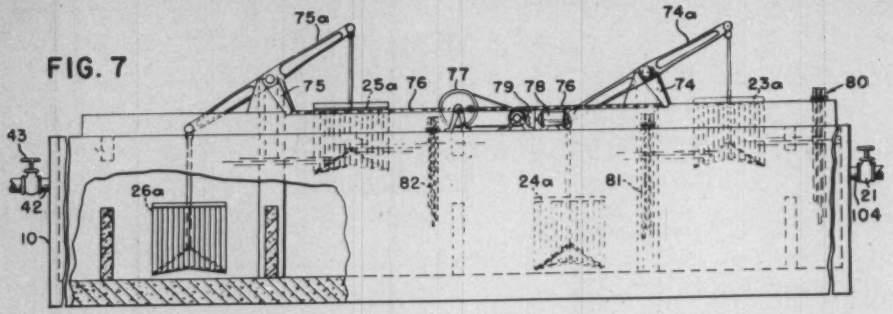
velocity wash water 46, so as to allow the bubbling of air or high-velocity water through the compartments.

The rectifier 31 is of any standard construction, with controls 47 and 48 for controlling the electrical energy, which results in the floc formation, in connection with the gate valve 21 which controls the rate of flow.

Structural Arrangement for Overhead Drive. The arrangement shown in Figures 7-10 differs from that of Figure 1 primarily in that the individual electrode units 23*a*, 24*a*, 25*a* and 26*a* are not stationarily mounted in the container 10 but are suspended on overhead-drive walking-beam arms, connected to levers 74 and 75 in such a manner that the electrode units suspended on each pair of arms are alternately immersed or withdrawn from the tank or mixing chamber. Each of the levers 74 and 75 has two arms 74*a*, 74*b*, 75*a* and 75*b* for holding the electrode units. To this end each lever is connected to an excenter disc 77 by means of a rod 76. The disc 77 is driven by a motor 78 through a speed transmission 79. It is obvious that rotation of the excenter disc 77 will impart a swinging movement to the levers 74 and 75, so that the latter will actuate the electrode units suspended thereon—referred to as the overhead-drive principle of electronic coagulation. The arrangement according to Figures 7 and 8 is provided with additional pipe sets 80, 81 and 82, through which additional chemicals may be added to the water if such should be required. It will be obvious that by moving the electrode units up and down (overhead-drive principle) in the body of water, the water is cut into slices of solution, while simultaneously the electrode units are kept clean. Otherwise, the operation of the overhead-drive electronic principle of Figures 7-10 will be the same as that described in connection with the operation of the arrangement illustrated in Figures 1-4.

Revolving Structural Arrangement. It has been found advisable to revolve the electrode units in the water to be treated, when the handling of water such as industrial wastes is encountered. Such an arrangement is shown in Figures 11-14. As will be seen, two electrode units 83 and 84 are suspended by means of a yoke 85 on a rod 86, which is supported by a thrust bearing, designated as 87 and encased in a casing 88. The upper end of the rod 86 is connected to a gear 89,

FIG. 7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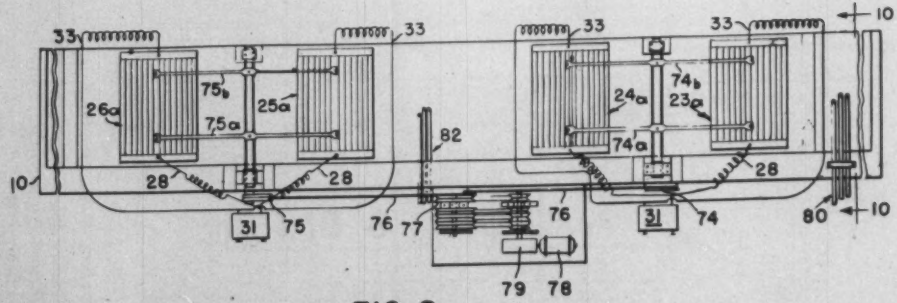
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FIG. 8

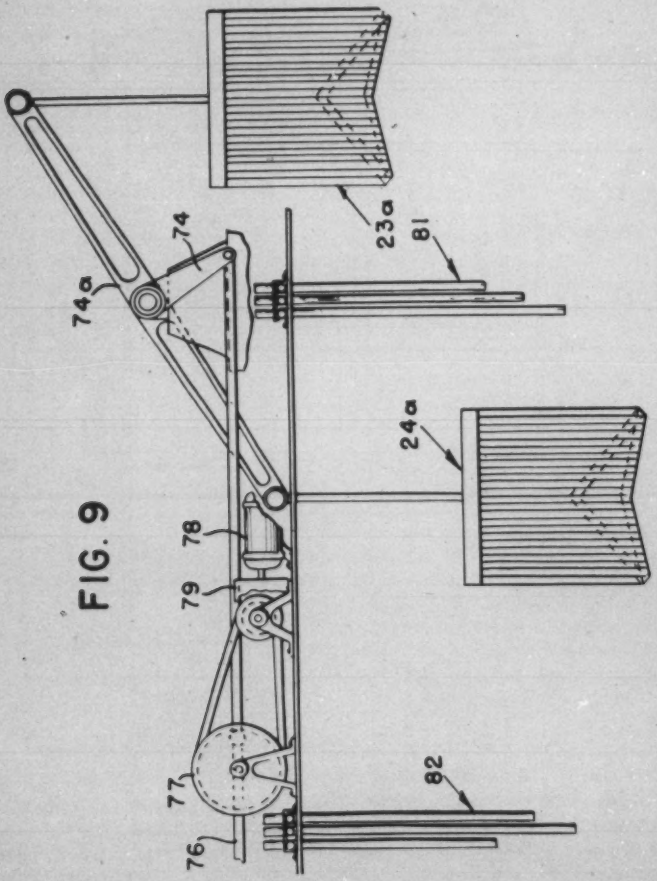
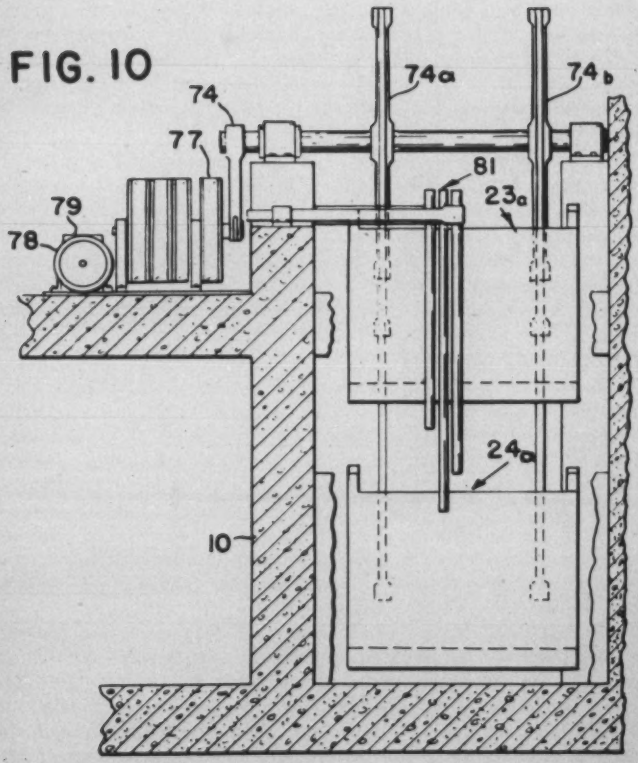


FIG. 9

FIG. 10



meshing with the gear 90 on a rod 91. The rod 91 is driven through a transmission 92 by a motor 93. The other electrode units are similarly arranged. The electrodes of the same polarity are connected to contacts 94 and 95 respectively, which in their turns are connected to contact rings 96 and 97. The contact ring 97 is adapted to be contacted by a contact 98, while the contact ring 96 is adapted to

be contacted by a contact 99. The contacts 98 and 99 communicate through wires 100 and 101 with a source of direct current—for instance, in the manner shown in Figure 1. The construction of the contacts 98 and 99 is the same and is illustrated in Figure 15. As will be seen, a spring 102 continuously urges the contact element 103 into engagement with the contact ring 96.

From the above, it will be clear that the operation of motor 93 causes the yoke 85 to rotate the electrode units in the water to be coagulated. In this way the advantages of mechanical flash mixing and mechanical movement of the colloids in the industrial waste-water are combined. As has been found out by numerous tests, a revolving electrical coagulating unit can be made to accomplish floc formation in water which is heavily loaded with pollution and industrial wastes. Holes may be provided in the electrode units, to give further contact with the water to be treated.

Summary. Instead of using the ion in filter alum, which is common practice in water-works color removal, with the electronic-coagulator principle the aluminum ion is removed from aluminum sheets by the use of direct-current electricity of low voltage and low amperage, the voltage and amperage varying according to the mineral content of the water. Water to be coagulated is passed through aluminum electrode units, connected to a source of direct-current electricity. This causes a practically instantaneous formation of a tough floc, which is not easily broken up and which can be removed from the water by settling and filtration.

During the operation of the electronic coagulator, the water is cut into slices of solution, because it has to flow between the control spaces of the aluminum sheets of the various electrode purifying units. The electrical energy from the plates is in continuous and controlled contact with the entire body of water passing through the units, as all of the aluminum plates contain the same amount of current and all of the water must pass through the spaces between the plates.

The mineral content of the water unites with some of the aluminum ions from the aluminum plates and, due to the electrical activity of the water, an aluminum compound is formed, which instantaneously forms an aluminum hydrous oxide floc, which removes organic color

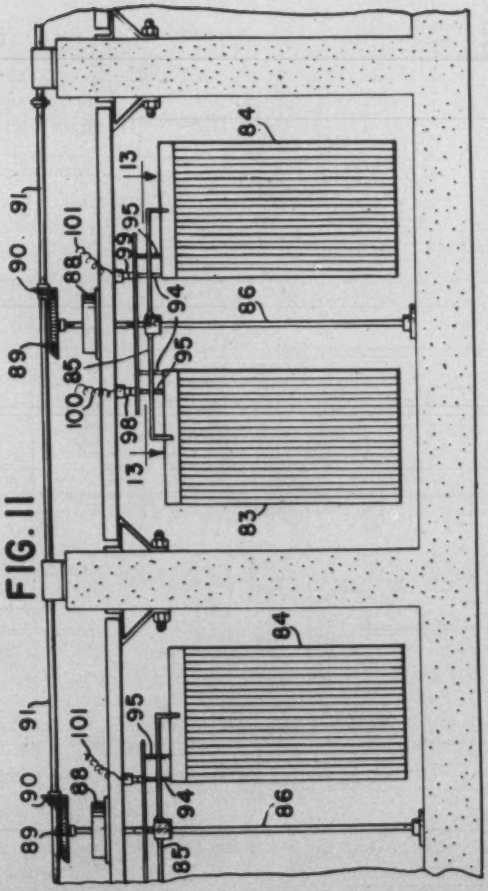

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FIG. 11

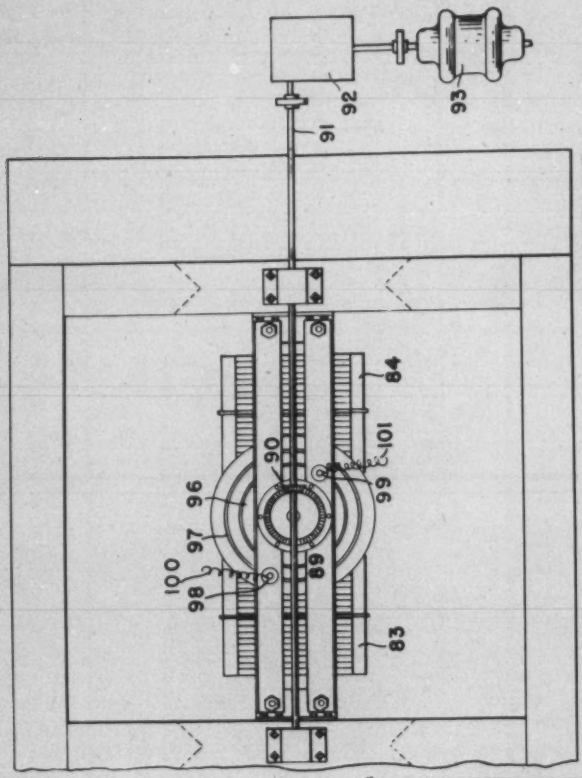

FIG. 12

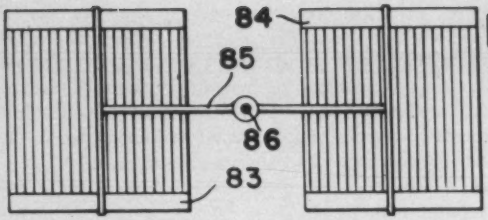

FIG. 13

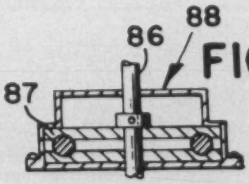

FIG. 14

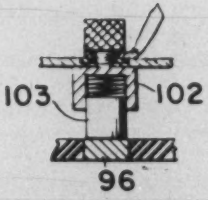

FIG. 15

from the water. The aluminum compounds can be flocculated out of the water by this electrical treatment without lowering the pH value, as is the case when applying filter alum and other acid-bearing coagulants, which depend upon the alkalinity of the water for floc formation. In the treatment referred to, hydrated lime may be added to the electronically coagulated water, to bring the pH value up to 9.2 without an increase in color, which cannot ordinarily be accomplished with present-day treatments of alum and lime.

It will be noted that one group of aluminum plates in each unit breaks down at a time—namely, the group from which the aluminum ions are split off. Therefore, in order to obtain even wear of the various electrodes in each unit, it is necessary to reverse polarity of the electrodes. Such reversal is accomplished by an automatic timer. The electrical energy and the amount of aluminum required for coagulation depend upon the organic content and the mineral content of the water to be treated.

Electronic purification does the coagulating job exactly as alum does but does it faster with less technical control. By dividing the water to be treated into slices of solution, velocity of coagulation is increased and, by the use of electricity, the production of floc formations can be accomplished within 45 seconds. Resulting floc is then passed to a settling basin or removed by gravity or pressure filters.

Discussion

Harold W. Griswold (Hartford, Conn.). On a supply of 100,000 gal a day, is that a practical way of flocculating the water?

Fred E. Stuart. Yes, sir.

Harold W. Griswold. With electricity at three or four cents a kilowatt?

Fred E. Stuart. Yes. It would not be as economical as the present practice, however.

CONSTRUCTION OF NORWALK LAKE DAM

BY JOHN E. RIORDAN*

[*Read May 16, 1946.*]

ALL water used by the system of the First Taxing District of the city of Norwalk comes from the runoff of the Silvermine River watershed, the total area of which is 10.4 sq mi. The watershed area is located in the towns of New Canaan, Wilton and Ridgefield, Conn., and Lewisboro, N. Y.

There are three reservoirs on the watershed: Grupe Reservoir in New Canaan and Brown and Scott Reservoirs, to the north, in Lewisboro, N. Y. The fourth, to be known as Norwalk Lake, is now under construction, with the dam located in New Canaan and the lake extending back into New York state. Flow from the upper reservoirs is along the stream bed of the Silvermine River to the intake at Grupe Reservoir. Figure 1 shows the locations of the reservoirs and Table 1 their areas, elevations and capacities.

TABLE 1.—IMPOUNDING RESERVOIRS

Reservoir	Water surface (acres)	Elevation of flow line	Total capacity (mil gal)
Scott	12.5	522.1	53.95
Brown	44.2	431.8	290.40
Grupe	22.0	293.7	55.70
Norwalk	80.0	370.0	550.00

The reservoirs on the watershed can be drawn upon during periods of low rainfall, to make up the difference between stream flow and consumption requirements. The present consumption is about 3.00 mgd, with a population of 20,000 served, or a per-capita consumption of 150 gal per day. This per-capita figure is high, due to the industrial usage of water, particularly by hat-manufacturing plants.

The yield of the existing system is 3 mgd during periods of extreme drought. However, many anxious days have been experi-

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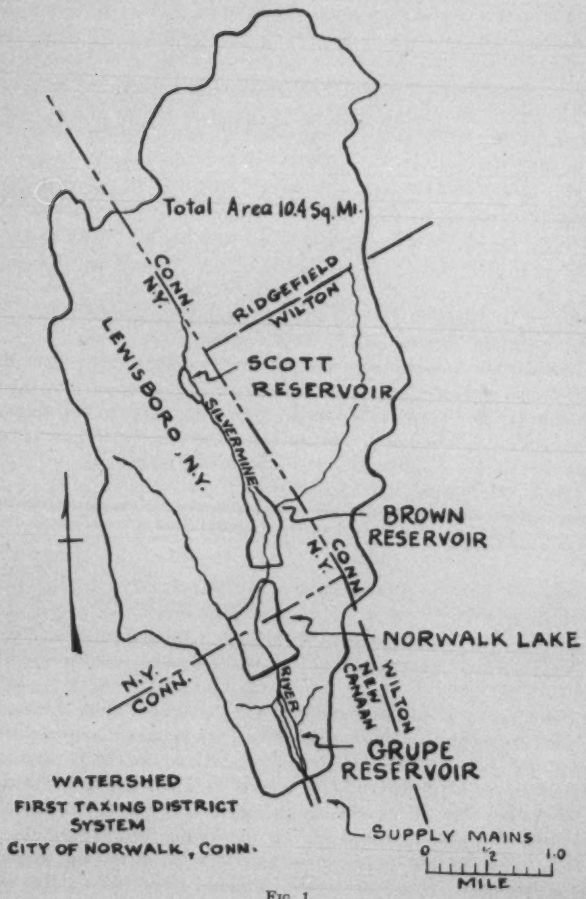
Total Area 10.4 Sq. Mi.
CONN. N.Y.
RIDGEFIELD
WILTON
SCOTT RESERVOIR
LEWISBORO, N.Y.
SILVERMINE
BROWN RESERVOIR
N.Y. CONN.
NORWALK LAKE
N.Y. CONN.
SILVER
NEW CANAAN
WILTON
GRUPE RESERVOIR
WATERSHED
FIRST TAXING DISTRICT
SYSTEM
CITY OF NORWALK, CONN.
SUPPLY MAINS
0
1/2
1.0
MILE

FIG. 1.

enced during the summers, dating back as far as 1931 and 1932. The storage in the Grupe Reservoir is not available, except under extreme conditions, as pressure must be maintained to supply consumers along the transmission lines and to furnish water by gravity to the filtration plant, 11,000 ft distant and at elevation 265.0. During 1943 and 1944, the storage was drawn down to a dangerously low level, so that only 30 days' supply remained when the situation was eased by rain. The drainage area tributary to the new reservoir will be 9.3 sq mi and the total yield of the system will be about 5.4 mgd, with a combined storage capacity of one billion gallons, or a year's supply based on present consumption.

Preliminary Work for New Dam and Reservoir

Surveys and topographical maps were prepared and land for the site of Norwalk Lake was acquired by condemnation proceedings in 1930. This involved about 100 acres of privately owned woodland in the Silvermine Valley, situated 8 miles northwest of the Norwalk business center. The average cost of the land was $400 per acre.

Clearing. The timber was cleared from the area to be flooded and sold as cord wood in 1935 and 1936. Stumps over the entire area were either pulled out by tractor or blasted by dynamite. Underbrush and second-growth trees will be cut to the ground line in the fall of this year, with completion scheduled in time for filling the lake. Brush and debris will be burned but no grubbing is planned.

Borings and Test Pits. Core borings were made in the latter part of 1944 and early part of 1945, at the location selected for the dam. This work was done by P. F. Beal & Sons of Brewster, N. Y., using steel shot drills.

Borings were made at intervals of about 75 ft on three lines across the site. One line was established as an axis, the second line was taken 75 ft upstream, and the third 75 ft downstream. All cores from the underlying bed rock were Thomaston granite. Wash samples taken at 5-foot intervals through the overburden across the valley were sand and gravel.

On the steeper parts of the east and west hills, test pits were dug by hand down to bed rock. The depth to rock on the hills was 10 ft and in the valley an average of 25 ft. The test pits revealed

a mixture of hardpan and gravel, with many large embedded boulders.

Exploration pits about the site showed the complete absence of suitable clay materials which might be used in an earth-fill dam. They did indicate extensive deposits of gravel in the northerly portion of the area to be flooded.

Design of Dam

The firm of Buck, Seifert & Jost, Consulting Engineers, of New York, N. Y., was engaged to design the dam. After considering an earth-fill dam with concrete core wall and several types of concrete dams, the mass-concrete gravity type was decided upon as best suited to the site and the materials available.

The concrete dam is designed with an abutment section against the steep west hill, a spillway 200 ft long across the central part of the valley and another non-overflow section up the gradual hill to the east. The over-all length at the top of the dam is 1,012 ft; the height above bed rock is 105 ft, and the height above ground line in the valley is 75 ft. A five-foot freeboard is provided, with spillway elevation at 370.0 and top of dam on the abutment sections at 375.0.

The width at the top is 9 ft and the width at the base in the deepest section is 77 ft. The upstream face is vertical and the downstream face is sloped, 7 ft horizontal to 10 ft vertical, from a point 7.5 ft below the top of the dam to the toe. A concrete parapet wall is set on the top of the dam at the upstream edge, for protection against wave action, and a pipe rail is set on the downstream edge. Figure 2 shows the maximum cross-section of the dam.

Spillway. The largest flow recorded to date on this stream was 2,000 cfs during the hurricane of 1938. The spillway is designed to discharge safely a flood flow of 8,000 cfs. The entrance lip of the spillway is elliptical in shape to the crest, where it changes to a parabola for about 10 ft on the downstream side and then follows a straight slope of 6.5 ft horizontal to 10 vertical. The straight slope emerges in a rounded concrete bucket, which directs the flow of water upward, to prevent scouring about the toe. Excavation in front of the spillway is arranged to form a stilling pool. Training walls are located at each end of the spillway.

Inlet and Outlet Works. The intake well, located at the easterly

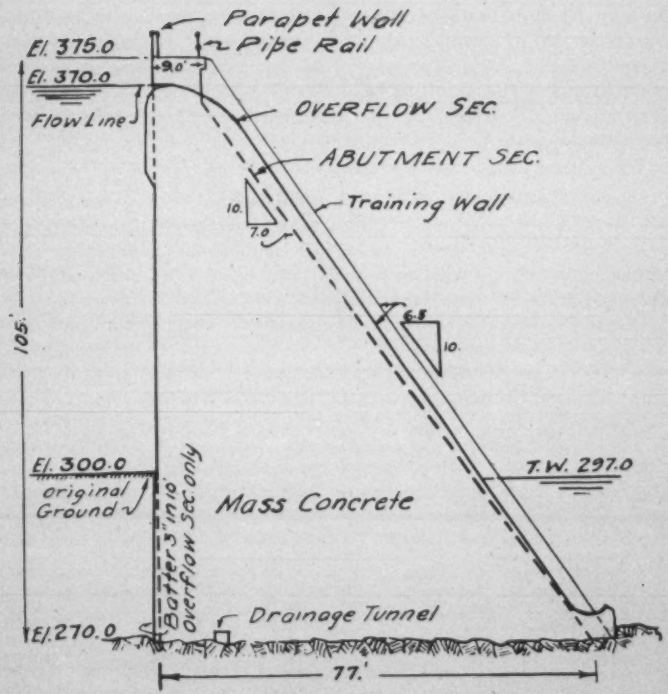

FIG. 2.—CROSS-SECTION OF NORWALK LAKE DAM.

end of the valley, is in the shape of a half-circle, built on to the upstream face of the dam. It is constructed of a 12-inch, heavily-reinforced concrete wall, landed on a concrete mat on bed rock, and rises to the level of the top of the dam. A cantilever concrete walkway is provided around the top of the intake, where the operating stands for the sluice-gates are located in the open. Circular sluice-gate openings are set at four levels, for selection of best-quality water.

One 24-inch, cast-iron outlet pipe runs from the intake well

through the dam to the downstream face, then turns west and discharges on to a riprap slope to the stream bed. Two outlet valves are set in vaults, for control of flow to the lower reservoir. A 24-inch tee is provided for a future pipe-line to be laid, by-passing Grupe Reservoir and connecting to the transmission mains for direct feed from Norwalk Lake.

Bids

Advertisements for bids were published in several newspapers and construction magazines. Bidding was open to all contractors meeting the requirements as to experience, financial ability and performance on jobs of this size and type. Bids were received from twelve contractors and the low bidder was E. W. Foley, Inc., of Brooklyn, N. Y. The contract, on a unit-price basis, was signed by the District Commissioners and E. W. Foley, Inc., on October 13, 1945. The range of bids varied 100% between the low and the high. Table 2 shows the items and unit prices in the contract.

Table 2.—Unit Prices and Estimated Costs of Various Kinds of Work in Contract for Norwalk Lake Dam

Item no.	Description	Quantity	Unit price	Amount
1.	Cofferdams and pumping, lump sum			$ 20,000.00
2.	Earth excavation and backfill	50,000 c.y.	$ 2.00	100,000.00
3.	Rock excavation	5,500 c.y.	5.00	27,500.00
4.	Class A concrete (less cement)	500 c.y.	22.00	11,000.00
5.	Class B concrete (less cement)	73,500 c.y.	7.00	514,500.00
6.	Steel reinf., if added	2,000 lb.	.07	140.00
7.	Water stops, installed	50,000 lb.	.23	11,500.00
8.	Drilling grout holes	300 l.f.	6.00	1,800.00
9.	Grout, furnish and place	200 c.f.	10.00	2,000.00
10.	Gravel (for drains)	300 c.y.	4.60	1,380.00
11.	Riprap	500 sq.y.	4.40	2,200.00
12.	Gates, valves and fittings installed, lump sum			9,200.00
13.	Miscellaneous metal work installed			6,000.00
14.	Road and paths	1,200 sq.y.	2.10	2,520.00
15.	Seeding	5 acres	500.00	2,500.00
16.	Cleaning up, lump sum			5,000.00
17.	Portland cement (in place)	80,000 bbl.	2.75	220,000.00
		Total		$937,240.00

Some items will be eliminated entirely and substantial savings on estimated quantities are expected to bring the cost below $900,000.

Construction

Excavation. Operations started in late October, 1945, with two large power shovels excavating the earth and boulders from the easterly section of the job. As bed rock was uncovered, the weathered and seamy rock was blasted out for an average depth of 4 ft, or until fresh, sound granite was found, regardless of depth. Excavated material was hauled to designated spoil areas on the property.

Stream Diversion and Cofferdams. Work proceeded from east to west and, as excavation reached the valley floor, the stream was diverted from its natural location in the center of the valley to the westerly end. This was accomplished by starting north of the site and cutting a canal, 22 ft wide and 4 ft deep, along the toe of the steep west hill and then back to the stream bed below the job. This left two-thirds of the length of the site free for excavating and placing of concrete. Work stopped December 15, 1945, due to the severe winter weather, and was resumed in the early part of March, 1946.

The overburden of sand and gravel was excavated to bed rock in the valley for a width of 80 ft at the bottom of the cut and 110 ft at the top in the deepest section. The inflow of water through this material was much less than had been expected. Two 4-inch centrifugal pumps, operating 8 hours daily, kept the excavation dry. The work was also protected from possible flood, by building cofferdams of earth, about 10 ft high and 50 ft wide, from the east hill, along the upstream and downstream edges of the excavation, and terminating against a low natural ridge on the east side of the canal. These embankments will also serve as elevated platforms for the power cranes while placing concrete. When concreting reaches the west side, the stream will be returned to its natural location and passed through an opening in the concrete, which will be filled after the intake is completed.

Concrete. A processing plant for the gravel to be used in the concrete was set up during the winter months, on the east side of the stream and 1,800 ft north of the dam. Both the sand and stone produced meet the specifications for quality and required sizes. The stone is stock-piled in three sizes, ranging from ¼ in. to 1¼ in., from 1½ in. to 3 in. and from 3 in. to 6 in. A minimum of cement is required, by using the large aggregates. Cement is handled in bulk

from the railroad siding at New Canaan to a 1,000-barrel storage silo adjoining the gravel hoppers. Closed-body trucks and enclosed bucket-type elevators move the cement with very little dust annoyance. The cement and gravel plant is shown in Figure 3.

Placing. The contractor eliminated a stationary concrete-mixing plant by using transit-mix drum-type trucks. The proper weights of sand, stone, water and cement are charged into the transit-mix trucks at the gravel plant and mixing takes place in the trucks on the run to the forms. Minimum mixing time of 3 min is required. The trucks mix a four-cubic-yard batch and, at present, six trucks are being used.

The concrete is discharged from the trucks into two, cubic-yard, bottom-opening concrete buckets, attached to long boom cranes. The cranes swing the concrete to the proper place in the forms, where it is moved and vibrated by heavy-type electrically driven vibrators.

Forms. The usual panel-type forms, common on this type of dam construction, are used over many times. One-inch tongue-and-groove matched lumber, with 2″ x 6″ studding, placed 12 in. on centers, is used with timber wales and steel angle-irons for stiffening on the outside. Patented steel tie-bracing, connected to bolts set in the concrete block below on each lift, prevents the forms from getting out of alignment as the concrete is deposited. As no plywood is available, the lumber for face concrete must be kept in good condition. Some of the forms are shown in Fig. 4.

Mix. A concrete technician controls the mix on the job. Absorption tests on the sand are run at frequent intervals, so that the amount of mixing water can be adjusted accordingly. Moisture absorbed by the stone is negligible. Trial mixes resulted in the following being used: cement, 1.11 bbl per cu yd; water, 6.5 gal; water-cement ratio of 0.57; slump, 1.5 in., and weight of concrete, 152 lb per cu ft. More economical mixes are expected, as the job progresses, by adjusting the sand and stone quantities and cutting the water content. Three test cylinders are sent to a New York laboratory daily. Twenty-eight-day strength averages 3,500 lb per sq. in.

Cooling. The concrete is placed in five-foot lifts and in alternate blocks of fifty feet. A waiting period of 21 days is required before the adjoining blocks are poured. On the same blocks a five-day interval is required between successive lifts. The waiting periods are re-

FIG. 3.—VIEW OF GRAVEL PLANT AND ELEVATED BULK-CEMENT BIN, WITHIN AREA TO BE FLOODED.

FIG. 4.—VIEW OF DAM SITE FROM EAST END.

quired for proper cooling of the mass concrete, so that excessive temperatures and cracking are avoided.

Drainage in Dam. Water stops of 3/16″ x 8″ wrought iron are used in keyways in all horizontal joints and 6″ x 12″ open vertical drains are placed in each 50-foot vertical joint, near the upstream face. The open vertical drains are connected to a gravel-filled drainage tunnel, running on bed rock the full length of the dam. The drainage tunnel discharges into a catch-basin and then to the stilling pool below the dam.

Progress. The job is still in the early stages but it is now doubtful that the contractor will complete the structure in the bid time of 400 calendar days, which would be about December 15, 1946. The concrete work is proceeding rapidly, pours of close to 600 cu yd per day being made. Rock preparation is slowing the concrete work. However, the dam should be of sufficient height by the end of the year to permit storing water and thus to prevent a shortage in 1947. Materials are obtained by ordering six months in advance of the required delivery date. Labor prices range from $1.25 per hour for common labor to $2.10 per hour for some skilled mechanics. The completion of this dam will consummate a long-needed addition to the water supply for Norwalk.

Acknowledgments. The resident engineers for the firm of Buck, Seifert & Jost are Edwin H. Hartman and Barton Jones. The geologist for the project is Chester R. Longwell, of New Haven, Conn. The members of the First Taxing District Water Commission, City of Norwalk, are William J. Powers, John D. Milne and Charles E. Sutton. Robert J. Lahey is District Clerk and the writer, District Engineer.

COMMENTS ON THE CONSTRUCTION OF THE SAUGATUCK DAM

BY SAMUEL P. SENIOR*

[*Read May 16, 1946.*]

THE Saugatuck dam is about 12 miles northwest of Bridgeport, with 35 sq mi of watershed above it. Our company, using its own equipment and personnel, built the dam, which is about 126 ft high over-all, 100 ft thick at base and about 1,100 ft long. Construction was begun in 1941 and finished the following year. The spillway is 299 ft long and 6 ft high, its elevation being 280 ft above mean high water. The reservoir stores 12 billion gal, is 4½ miles long and has a water area of 870 acres.

The work required relocating and rebuilding about 7 miles of highways, with bridges, culverts and standard highway-department wire-rope railings. Wood and timber were cut from the basin and the stumps pulled out for a distance of 20 ft below the flow line. A concrete dike, with a maximum height of 40 ft and a length of 800 ft, was built in a low spot on the easterly rim of the reservoir, to prevent an overflow in that direction.

Gravel for the aggregates was secured about 3 miles up the valley, where the company built a modern crushing, screening and washing plant, which separated the gravel into three sizes. The sand and gravel were both thoroughly washed. Twenty-three per cent of the aggregate in each batch was 2″ to 6″ gravel, 18% was 1¼″ to 2½″ gravel, 30% was ¼″ to 1¼″ gravel and 29% was sand. The concrete was mixed very dry, with a ½-in. slump. In the interior of the dam, 0.93 of a barrel of cement was used, while near the face, 1.07 was used. The concrete was transported from the mixer out over the dam by a Lidgerwood cableway with a 15-ton capacity, having a movable tail tower, by means of which the concrete was deposited in all parts of the dam.

Volume changes in the concrete were kept at a minimum by using

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low-heat cement—Federal specification S-S-P 206—while strength was assured by using a stiff mix, having a low water-cement ratio of 0.54. The concrete had an average compressive strength of 4,600 at 28 days and one of 5,700 in one year. The Scientific Concrete Service Corp. furnished specialized equipment, which was located on the batching floor and operated throughout by one of its technicians. A graphic recorder, attached to the scale head, made a permanent record of the aggregate in each batch. Moisture-determination equipment, bank control levers for actual weighing and controls for operating the concrete mixer were located on the batching floor. Moisture-determination measurements were so nearly automatic that they were very frequently made, compensation being provided for different moisture contents of the sand and other aggregates.

Because of the small slump, vibrators were attached to the 4-yd buckets, in order to dump them. Barrel-type pneumatic vibrators, weighing 90 lb each and operated by two men, were used to consolidate the concrete in the forms.

Concrete was placed in 15-in. layers and in blocks averaging 4 ft high and 40 ft long, extending the entire width of the dam. Three days' elapsed time was required between lifts. Following our usual practice, we filled in earth against the downstream face of the dam to a height of 235 ft above mean high water, which is within 45 ft of the spillway level. This was done solely to protect the most vulnerable part of the dam from freezing and spalling.

It took from January 27, 1942, when the waste gates were closed, until December 17, 1942, for the reservoir to fill to overflowing. Meantime, beginning in July a little more than 1 billion gal was drawn off for use. In addition to this, we are required to keep a minimum flow in the river below the dam of 4 mgd. This would account for a little over a billion more gallons during the year required for filling.

The total cost of all concrete placed in the main dam, aggregating 79,000 cu yd, was $6.25 per cu yd. In this price was included erecting all construction buildings, water supply, dismantling plant, sand-and-gravel plant, cement-unloading plant—where bulk cement was unloaded by a screw from the bottom of hopper cars into a steel standpipe—mixing concrete, placing concrete and all forms. Included in

this price also was the cost of the cableway, mixing plant, crushing plant and all other equipment used, less the price for which it was sold—namely, about one-half of the original cost.

The Saugatuck reservoir is not a distributing reservoir. It stores water for use in the Hemlocks reservoir—elevation, 225 ft. The water is delivered through two tunnels and a 48-in. cast-iron pipe. Two ridges intervene between the Saugatuck and Hemlocks reservoirs. A tunnel, 1,900 ft long, was driven under one ridge and one 5,900 ft long was driven under the one to the east, near the Hemlocks distributing reservoir. These tunnels were 6 ft 4 in. inside of the 12-in. concrete lining. The rock was quite hard and required little timbering. Each of these tunnels was driven from a portal at one end. Gardner-Denver mucking machines loaded muck on cars, pulled by electric-storage-battery locomotives. A non-telescopic steel form was used for placing the concrete in the tunnel. A mixer, located over the portal of the tunnel, discharged concrete into 1-yd bottom-dump hopper cars. The hopper cars ran up on a low bed rig, which had a belt conveyor for carrying concrete to its head, where it discharged into a ½-yd pneumatic concrete placer. The placer was used to shoot concrete through a 6-in. pipe to final location behind the forms. A 2-in. slump was used for the invert section, with about a 4-in. slump for the arch concrete. The water-cement ratio for both invert and arch mixes was 0.56 by weight and a compressive strength in excess of 4,000 lb in 28 days was obtained.

As we go to see the Saugatuck reservoir today, we shall pass for several miles along the west side of the Hemlocks reservoir, from which the Saugatuck water is distributed, as described above. At the upper end of the Hemlocks reservoir you will see how the water is received through an aerator, which throws it up to a height of 35 ft.

EXPERIENCE WITH BREAK-POINT CHLORINATION IN CONNECTICUT

BY FREDERICK O. A. ALMQUIST*

[*Read May 16, 1946.*]

CHLORINATION was said to have turned a corner in 1939, when so-called "break-point chlorination" was introduced. When one considers the progress in chlorination for the disinfection of water, since it was first reputed by some authorities to have been used in Boonton, N. J., in 1908, chlorination has evidently turned several corners. In looking back, we note in the various stages the addition of chlorine to obtain a small residual—known as "marginal" or "normal" chlorination — super-chlorination with de-chlorination, chlorination with ammonia to form a chloramine and now break-point chlorination. When one considers the uses for chlorination of water, such as disinfection, taste and odor control, control of algal growths, bleaching and color reduction and improvement in connection with filter operation, among others, one has a great deal of respect for the simple chemical element known as chlorine.

When a process is relatively new and shows promise, a great deal of enthusiasm accompanies such a process and occasionally, as history has shown, the benefits are not at first entirely known and are oftentimes exaggerated. Sometimes it is desirable to review and weigh the various aspects, so as to obtain a more complete picture. It is the writer's purpose in this paper briefly to review the experiences with break-point chlorination in Connecticut, for the information of water utilities not now employing break-point chlorination but contemplating its use. It is the writer's understanding that M. S. Wellington, sanitary engineer, and Dr. Samuel Jacobson, chemist and bacteriologist, of the New Haven Water Co., are preparing a paper on experiences with break-point chlorination at New Haven, which will detail the actual process of break-point chlorination. The writer will therefore not go into this phase. Suffice it to say that by break-point

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chlorination the writer means the addition of chlorine to a point where free available chlorine is produced and maintained for a period of time in a water supply.

Although break-point chlorination has been known and used for several years, it has been only within the past two or three years that it has been practiced in Connecticut. There are now 13 water utilities which are or have been using planned break-point chlorination on 26 supplies, although our inspections show many more reach break-point chlorination at certain times of the year, under certain conditions.

The writer has tabulated, in the course of a number of chlorinator inspections of Connecticut public water supplies, the flash test and the five-minute readings after a ten-minute contact period has been allowed and, during the winter season with low chlorine demands and low temperatures, free available chlorine is frequently maintained, sometimes carrying well into the system. The free chlorine has been such that break-point chlorination was actually in practice. These results were obtained on surface-water supplies, where only normal chlorination was planned, and invariably, when the chlorine demand increased with warmer weather, the free chlorine disappeared in favor of chloramines.

One of the earliest claims for break-point chlorination was its elimination of taste and odors. However, it was soon found that the improvements in bacteriological results were more important and certainly, so far as health workers are concerned, this is the most important consideration. The writer would like to discuss improvements in bacteriological results, comment on taste and odor control and, finally, call attention to practical aspects in the operation of break-point chlorination.

Bacteriological Improvement

The writer has obtained permission from V. O. Ketcham, president and general manager of the Stamford Water Co., to use the bacteriological analyses of water samples collected from this system and has drawn freely on this source for material. The analyses used are those made by the Bureau of Laboratories of the Connecticut Department of Health, although a large number of additional samples

is examined in the laboratory of the consulting chemist. Ralph Keeler, the assistant to the general manager, has had direct supervision over the treatment and has supervised the treatment in a most meticulous manner.

The source of water supply for the Stamford Water Co., located in the southwest corner of the state of Connecticut, is a series of reservoirs on the Mill River. The watershed area above North Stamford Reservoir, the lowest reservoir, is about 21.6 sq mi and the total storage is relatively large. The watershed is moderately populated, with no source of gross pollution, although, because of surface drainage, coliform organisms are frequently isolated in the untreated water. The physical characteristics of the water are good, with a normal color of probably less than 20. In spite of close algae control in the distribution reservoir, abnormal tastes and odors occur occasionally in sufficient intensity to cause complaints. North Stamford Reservoir is the distributing reservoir and water is conveyed to the distribution system a distance of about 5.5 miles by gravity. The supply mains are three in number—a 36-inch cast-iron main, a 24-inch cast-iron main and a 16-inch cement line. These are interconnected at several points on the way to the distribution system.

The Stamford Water Co. was an early pioneer of chlorination in Connecticut. In fact, it was the first water utility in Connecticut to install a chlorine machine, the machine being installed in 1913. Later, when chlorine and ammonia treatment was developed, this treatment was started on May 4, 1931. With the advent of break-point chlorination, ammonia treatment was discontinued in favor of break-point chlorination. This was done under the recommendation of the Newlands Sanitary Laboratory of Hartford, consulting chemists for the Stamford Water Co. Break-point chlorination was started on December 29, 1943, but was discontinued December 28, 1944, until August 2, 1945, when it was resumed. The normal dosage of chlorine is in the vicinity of 1.0 ppm, with break-point chlorination at 2.0 ppm or slightly more. Unfortunately, the capacity of the pumping equipment on the chlorinator installation limited the rate of chlorine dosage to about 2.0 ppm under normal maximum rates, which did not furnish as much free chlorine as would have been desired. Because of the difficulty in maintaining the rate of chlorine dosage as high as desired

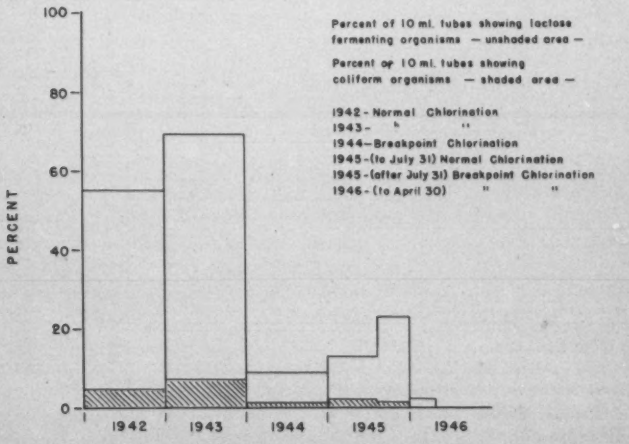
with maximum rates of water consumption and because abnormal tastes and odors of a chlorinous nature still persisted, break-point chlorination was discontinued and normal chlorination resumed on December 28, 1944. On July 18, 1945, the rate of chlorine dosage was increased to 1.5 ppm and then gradually increased to 2.0 ppm on August 2. Chlorine-residual tests, made just beyond the point of chlorine application after a 10-min contact period is allowed, have been between 1.2 ppm and 1.5 ppm, with the flash test averaging from 75 to 100% of the 5-min reading. In order to be in a position to increase further the rate of chlorine dosage, the Stamford Water Co. is contemplating larger-capacity equipment in the near future. This will enable the installation to furnish 100% free chlorine at times of maximum draft.

During the period of changing the type of chlorination, a greater number of samples was collected for bacteriological analysis than would ordinarily be the case. Control points were established, where weekly chlorine residuals were taken and samples collected for analysis. As the chlorine residuals were established in the distribution system, several of the points were moved. At each of the points a record was kept of chlorine residuals, both flash and 5-min readings, in addition to the collection of the samples. The bacteriological results are given in Table 1. "Standard Methods of Water Analysis" was followed in the analysis, 5 10-ml portions being used and the tests being carried through to completion.

TABLE 1.—RESULTS OF BACTERIOLOGICAL EXAMINATION OF WATER SAMPLES FROM STAMFORD WATER CO.

	1943	1944	1945 (1/1-7/31)	1945 (8/1-12/31)	1946 (to Apr. 30)
No. of samples examined	20	131	75	52	39
No. of 10-ml tubes	100	655	375	260	195
No. of 10-ml tubes showing lactose-fermenting organisms	69	55	50	60	4
% of tubes showing lactose-fermenting organisms	69	8.4	13.3	23	2.1
No. of tubes showing coliform organisms	7	11	8	5	0
% of tubes showing coliform organisms	7	1.7	2.1	1.9	0

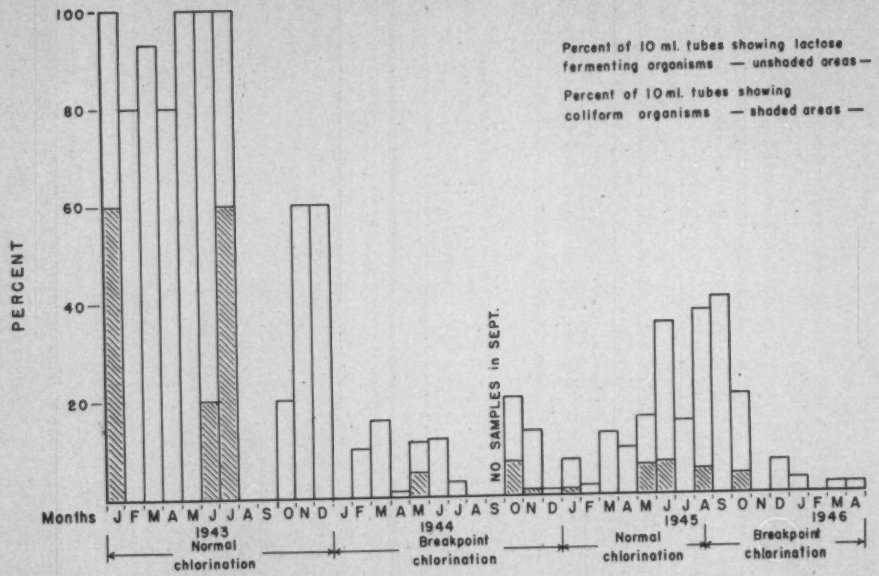
The reduction in the number and percentage of lactose-fermenting organisms as well as coliform organisms between the years 1943 and 1944 is shown. The difference is not so striking between the first portion and the second portion of 1945, although there is a reduction in the coliform-type organisms. It was stated that a very heavy run-off occurred toward the end of July and beginning of August in 1945, which affected the results and retarded the expected results from the return to break-point chlorination at that time. However, a considerable difference is noticeable, with the continuance of break-point chlorination, in the analyses during the past six months. The same results are shown by years and also by months in Figures 1 and 2, respectively. This only confirms the findings of others that break-point chlorination results in improved bacteriological conditions in the distributing system, after the free chlorine has been established through the system.



BACTERIOLOGICAL RESULTS IN STAMFORD WATER Co. SUPPLY

STAMFORD CONN.

FIG. 1.


Percent of 10 ml. tubes showing lactose fermenting organisms — unshaded areas —
Percent of 10 ml. tubes showing coliform organisms — shaded areas —
PERCENT
100
80
60
40
20
NO SAMPLES in SEPT.
Months J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A
1943
1944
1945
1946
Normal chlorination
Breakpoint chlorination
Normal chlorination
Breakpoint chlorination

BACTERIOLOGICAL RESULTS IN STAMFORD WATER Co. SUPPLY

FIG. 2.

Tastes and Odors

Tastes and odors are very closely allied. It has been said that 90% of what is normally described as a taste is in reality an odor. Yet there seems to be a sufficient difference so that we may treat them separately. The addition of increasing amounts of chlorine to a water containing ammoniacal or organic products produces a chloramine, until the combination has taken place or the reaction has been completed; then the organic material is oxidized and finally free chlorine is found. In passing through these stages, tastes and odors are normally produced until the free chlorine remaining is upward of 100% of the total chlorine. The intensity of the tastes and odors varies considerably with the type of water encountered. In most instances when 100% free chlorine remains, the so-called taste seems to disappear. By "100% free chlorine" the writer means that the 5-min reading has not increased over the flash-test reading. The so-called odor in the water, particularly in the distribution system, however, does not disappear as quickly as the taste and frequently an odor is present, the odor being chlorinous in nature. In many cases we have found that the odor will decrease quickly to almost zero, when water is drawn from a tap. This occurs quickly enough so that, if the threshold-odor test is made, the results are probably erroneous. If a method could be devised to fix the odor as it appeared in the water immediately on leaving the tap, the threshold-odor test would have more value. Although, in most instances, the chlorinous odor leaves quickly after the water is drawn from the tap, it is during this short period of time, when first turning on the tap, that the householder notices the odor and voices his objection.

Objectionable tastes and odors are obtained in at least portions of the distribution system, while break-point chlorination is being established. As the free chlorine penetrates into the distribution system, a border or band of varying width fans outward, in which chloramines are formed between the chlorine and the organic matter in the piping system. This band may move rather slowly and, in the Stamford system, it took about one year to travel approximately 6 mi. Here a rate of dosage of about 2 ppm of chlorine was used and free chlorine varied from about 75 to 100% during that time. This is just beyond the point of chlorine application. When samples were tested

for chlorine residual within this band, chloramines were found and complaints were received because of abnormal tastes and odors. It is of interest also that many times, when samples were collected for bacteriological analysis within this band, not only were lactose-fermenting organisms found but confirmation as organisms of the coliform group was obtained in some instances. This was true, despite the fact that chlorine residuals of 1.0 to 1.5 ppm after a 10-min contact period were maintained at the chlorine plant.

Occasionally an algal growth may develop under the ice, where difficulty is encountered in treating with copper sulphate. In at least one instance, it was felt that the break-point treatment had eliminated the abnormal tastes and odors that would have been experienced in the distribution system, as a result of the algal growths. This should be a decided advantage, since it is expensive and sometimes almost impossible to eliminate algal growths under a good thickness of ice.

When investigating complaints of abnormal tastes and odors in distribution systems, it would be well to take into consideration local conditions, such as old service pipes. A reaction between the chlorine and material in an old service pipe might be expected to affect the water in the house for some time, depending on the condition of the service pipe.

Occasionally odors and presumably tastes have been found in untreated water, which break-point chlorination was not able to remove, even with relatively high dosages of chlorine. It was evident from our field studies that break-point chlorination is not a panacea for taste and odor troubles. The writer was told, however, on several occasions in different supplies that, although a chlorine odor seemed to be present, the general improvement over the previous normal chlorination was appreciable.

Practical Aspects in the Operation of Break-point Chlorination

After the foregoing discussion of bacteriological improvement and tastes and odors in general, it would seem desirable to mention a few points with regard to the practical application of break-point chlorination.

Initiation of Break-point Chlorination. If a water utility makes the decision to establish break-point chlorination, in order to improve bacteriological results in the distribution system or to improve the physical characteristics of the supply, there are two or three considerations which should be kept in mind. First, abnormal tastes and odors will undoubtedly result in the distribution system, until the organic matter in the system has been dissipated or oxidized and free chlorine established. This will depend on the condition of the interior of the piping system. Secondly, some material may be expected to loosen from the interior of the piping. For this reason, the rate of chlorine dosage should probably be increased slowly over a period of several weeks to two or three months. Flushing will probably be necessary in most cases. Thirdly, it would seem logical that less disturbance might be expected, if break-point chlorination were started during the cold-weather months, when the organic content of the water is usually lowest and some free chlorine is already penetrating farther into the distribution system than during the summer season. The lower temperature of the water tends to lessen abnormal tastes and odors at this time and, in addition, the problem is not usually further complicated by possible algal conditions in a reservoir.

It is likely that, once break-point chlorination is established, it should be continuous and not intermittent. However, if this method is stopped for a period of time and the period of time is not too great, the return to break-point chlorination the second time or succeeding times may not be as noticeable as the first time.

Establishment of Rate of Dosage. Since every supply must be treated differently, no hard and fast rule seems available as to the establishment of a proper rate of dosage. Jar tests appear to be desirable for the establishment of the proper rate. If the making of jar tests is not feasible, use of the Laux "flash-point test" can be made to set the dosage. The chlorine dosage can be increased and chlorine-residual readings observed immediately and at the end of a 5-min period. When a point is reached where the flash test does not increase at the end of 5 min, this is probably the proper rate of dosage. Observations can then be made for possible tastes and odors at the chlorinator and also in the distribution system. Observations,

so far, have indicated that the maintenance of 100% free chlorine, rather than 75% to 90% suggested earlier, is desirable. However, when jar tests are used, water can be held for varying lengths of time and the chlorine-residual tests supplemented by threshold-odor observations, which are more sensitive and perhaps a little more scientific. When jar tests are used, it would seem that they should be made quite often at first, perhaps with each change in run-off conditions, until the necessary frequency of making the tests is established. In other words, if the break-point has been established at a certain residual, it is possible that some changes in run-off conditions will not affect the results.

Capacity of Equipment. If it is decided to employ break-point chlorination and the possible necessary dosage is established, the next step would seem to be to check the chlorine equipment, to determine whether the necessary capacity is available. Fortunately the capacities of most chlorine machines can be increased considerably by changing orifices and scales or the meter tubes. However, when a rate of chlorine dosage of 1 ppm or less is increased to 2, 3 or even 4 ppm, it is often necessary to make additional changes in the machines or even replace them with new ones. When pumping equipment and injectors are used, it may be necessary to enlarge the pumping-equipment capacity. In Connecticut we do not approve chlorinator installations depending entirely on a single source of electric power. Where it is desired to provide electrically driven pumps, to save water lost in operating hydraulically operated pumps, it is necessary to have available hydraulic pumps as stand-by equipment. Under these conditions automatic devices must be provided, to start the hydraulic pumps upon electric-power failure, which would cause the electric pumps to become inoperative. It would be well to discuss capacities of both chlorinator equipment and pumps with the manufacturer, before beginning break-point chlorination.

There will probably be a considerable increase in the amount of chlorine used, especially by some of the larger supplies, and some thought should be given to the extra handling and storing of chlorine cylinders.

Period of Contact Important. Field observations would indicate that the time of contact, when break-point chlorination is employed,

is important. In several instances, after break-point chlorination had been well established, taste and odor observations made a short distance—up to even two or three miles—from the point of chlorine application have indicated abnormal tastes and odors, whereas observations made at twice this distance have shown improvement. The added time of contact improved the physical character of the water. These observations could be checked by jar tests, although the latter are not entirely comparable to results in a piping system.

High Chlorine Residuals Advantageous. Break-point chlorination, with its resulting high chlorine residuals in most cases, has a distinct advantage over normal chlorination, where a sudden change in chlorine demand may occur. Such changes may be due to algal growths in a reservoir, heavy run-off with a relatively short period of storage, a change in wind conditions over a reservoir or reservoir overturns, among other conditions. With normal chlorination, accompanied by 0.20 or 0.30 ppm residual chlorine after a 10-min contact period, a sudden increase in chlorine demand would lower the chlorine residual, leaving the water under-chlorinated, and might even eliminate or dissipate all the residual. Studies of distribution systems in Connecticut have shown that, with under-chlorinated water, organisms of the coliform group enter the distribution system and, unless a chlorine residual penetrates to destroy them, they remain for a long period of time. Samples collected in the distribution system during this time may be poor bacteriologically, even though normal chlorination is maintained. In connection with the collection of samples for bacteriological analysis in the distribution system, it seems desirable to collect these samples with sodium thiosulphate, especially if the water contains a chlorine residual.

High chlorine residuals from break-point chlorination would be less affected by sudden increases in chlorine demand and would probably not be dissipated entirely, except under extreme conditions.

Pre-chlorination to Break-point. At one or two of our filtration plants, pre-chlorination has been used to break-point and a good residual carried through the filters. The presence of chlorine has been reported to be beneficial in aiding "floc" formation and there appears to be little question but that a cleaner filter and better performance are obtained, when chlorine is applied ahead of a filter and

carried through the filter bed. In starting, however, close observation should be made of the physical character of the filtered water, particularly if the filter is dirty. Unloading may take place, which will discolor the filtered water and cause abnormal turbidities, and a highly disagreeable taste and odor may result. The chlorine dosage should preferably be increased gradually and post-chlorination maintained, if not all the time, at least until a continuous chlorine residual can be obtained in the filter effluent at all times.

Need for Improved Chlorine-Testing Equipment. Testing for chlorine residuals in the field at chlorine plants where no laboratories are available is nearly always done with a testing outfit utilizing a comparator block and colorimetric comparison between a standard—either a liquid or a permanent glass disc—and the sample. The outfits in general use are quite satisfactory for normal chlorination but are not quite so satisfactory where break-point chlorination is used and high chlorine residuals are maintained. Several of our Connecticut supplies maintain chlorine residuals of 1-2 ppm. It is difficult to read the standard ortho-tolidin tests in this range and the writer feels that there is a need for an improved standard.

The small amount of interference normally encountered in Connecticut supplies with the ortho-tolidin test is usually negligible at break-point chlorination. However, in the distribution system, where lower chlorine residuals are maintained, a test for possible interference is desirable. It has been recommended to all public water supplies in Connecticut that a sodium arsenite solution be used, to determine whether any interference exists.

Bleaching Effect of High Chlorine Dosages. The amount of chlorine added to water to reach break-point chlorination will have some effect, because of its bleaching action, both on chemical tests and the physical color, under some conditions. This has been noticeable in the test for alkalinity, where methyl orange indicator is used. Three drops of methyl orange indicator, required under "Standard Methods of Water Analysis", will bleach out. It may be possible to maintain a color, if more indicator is added. A drop of weak solution of sodium thiosulphate, added before the indicator, would serve to remove the chlorine and would affect most tests very little. This bleaching effect is noticed in using certain indicators for pH-value

determinations. Whether other tests are affected has not been called to the attention of the writer. Apparently phenolphthalein indicator is not affected by reasonably high chlorine dosages.

The literature indicates that, with break-point chlorination, some bleaching of the color of the water has been experienced. While there has been some apparent bleaching effect on one or two supplies in Connecticut, the dosages of chlorine have not been of such magnitude that there has been anything startling with regard to color reduction.

Conclusions

In conclusion, the writer would like to emphasize again that each supply presents a different problem and needs individual attention. Experiences obtained on other supplies should be used only as a guide and not copied without investigation of local conditions.

While the enthusiasm for break-point chlorination is high in many quarters, exorbitant claims for it should be weighed. It is quite certain that break-point chlorination has decided advantages, from the standpoint of bacteriological conditions in the distribution system. Other advantages are not so definite and vary with conditions. Careful thought should be given before break-point chlorination is attempted and, if it cannot be supervised fairly closely, it might be well to employ the services of a consulting chemist or engineer, until the break-point process is established.

The writer in no way wishes to discourage the use of break-point chlorination. On the contrary, from the standpoint of a health-department representative, the fact that bacteriological improvement of samples of water collected in a distribution system appears certain should be sufficient to encourage the use of break-point chlorination. The writer has intended only to point out possible aspects that should be investigated. There are, under most circumstances, other improvements from the use of break-point chlorination, not the least one being improvement of the physical characteristics of the delivered water under most conditions.

CORROSIVENESS OF WATER TO METALS

PART II. PRACTICAL APPLICATION OF THE THEORY

BY THOMAS R. CAMP*

[*Read September 19, 1946.*]

INTRODUCTION

IN PART I† of this paper it was shown that all of the reactions in the corrosion of metals in contact with water, including those resulting in the formation of solid corrosion products, can be explained in terms of half-cell reactions, accompanied by the flow of electric current. In Table 1 of Part I is listed a large number of the half-cell reactions which may conceivably take place in the corrosion process, together with the corresponding standard electrode potentials of these reactions. A method was presented by which these reactions may be examined in pairs for a particular water and metal, to determine which reactions will take place and what corrosion products will be formed. No claim is made that Table 1 contains all conceivable reactions and there may be others, particularly with organic compounds, which play important parts in the process of corrosion.

In Part II it is proposed to give some of the results of the practical application of the theory which are of interest to the water-works profession, with particular emphasis on the corrosiveness of New England water. An analysis has been made to determine the principal reactions which take place in the corrosion of iron in fresh water. A study of these reactions indicates the relative importance of changes in pH value and the relative importance of the dissolved constituents in the water. The relative effectiveness of various methods of corrective treatment is also indicated. Since lead, copper and brass are widely used in consumers' services and plumbing systems, the analysis has been extended to determine the reactions which prevail in the corrosion of these metals and to determine the effect of corrective

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treatment on such corrosion. The analysis has been made in terms of the single-electrode potentials at 25° C. and the activity coefficients have been assumed to be unity in all cases. For greater precision, corrections must be made for the effect of temperatures other than 25° C. and for the activity coefficients.

Corrosion of Iron in Fresh Water

The principal reactions which take place in the corrosion of iron in fresh water are shown in Table 2. Corrosion proceeds by a transfer of negative electrons at anodic areas from the water to the metal. These electrons flow through the metal to cathodic areas, where they are given up to constituents in the water. This flow of electrons constitutes a flow of electric current and the circuit is completed through the water by the motion of ions between the two electrodes of the cell. In order that the electric current may flow, there must be an electrochemical reaction at the anode and an equivalent electrochemical reaction at the cathode. The two reactions which take place in a particular case are those which produce the greatest potential between the electrodes. The actual potential is determined by the standard electrode potentials, values of which are given in the tables, and the concentration of the ions and dissolved substances which take part in the reactions.

Principal Anodic Reactions. In conventional literature on corrosion, the principal anodic reaction in the corrosion of iron is the solution of metallic iron to produce ferrous ions, reaction 74 of Table 2. In this study it has been found that the solution of metallic iron may be prevented, if the water contains a sufficient concentration of hydroxyl, carbonate or phosphate, and that the corresponding anodic reactions will produce a solid coating, consisting respectively of ferrous hydroxide, ferrous carbonate or ferrous phosphate. Corrosion will proceed until the solid coating completely covers the metallic iron, after which corrosion must stop, unless the coating itself can take part in another anodic reaction. It may be seen, therefore, that with favorable constituents in the water the corrosion of iron may be inhibited by anodic protection. It is of interest now to determine the concentrations of anions required for the anodic protection of iron against corrosion.

TABLE 2.—CORROSION OF IRON IN FRESH WATER

		$E°_{25}$, volts
Principal Anodic Reactions		
36.	$2\ OH^- + \underline{Fe} \longrightarrow \underline{Fe(OH)_2} + 2e^-$	+0.877
51.	$CO_3^{--} + \underline{Fe} \longrightarrow \underline{FeCO_3} + 2e^-$	+0.755
—	$HPO_4^{--} + \underline{Fe} \longrightarrow \underline{FeHPO_4} + 2e^-$	?
74.	$\underline{Fe} \longrightarrow Fe^{++} + 2e^-$	+0.440
Principal Cathodic Reactions		
100.	$H_{2_{gas}} \longrightarrow 2H^+ + 2e^-$	+0.000
134.	$2\ CO_3^{--} + H_2O \longrightarrow 2\ HCO_3^- + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.66
144.	$CO_3^{--} + H_2O \longrightarrow H_2CO_3 + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.779
198.	$H_2O \longrightarrow \frac{1}{2}\ O_{2_{aq}} + 2H^+ + 2e^-$	—1.273
220.	$\underline{Fe(OH)_3} \longrightarrow \frac{3}{4}\ O_{2_{aq}} + Fe^{++} + \frac{3}{2}\ H_2O + 2e^-$	—1.386
Principal Cathodic Reactions in Chlorinated Water		
158.	$Cl^- + 2\ OH^- \longrightarrow OCl^- + H_2O + 2e^-$	—0.878
233.	$Cl^- + H_2O \longrightarrow HOCl + H^+ + 2e^-$	—1.49
240.	$3\ Cl^- + \underline{Fe(OH)_3} \longrightarrow 3\ HOCl + Fe^{++} + 5e^-$	—1.58

A study of reactions 51 and 74 indicates that at pH 7 the bicarbonate alkalinity must exceed 454 ppm to prevent the solution of iron, if the concentration of iron already in solution is 0.3 ppm. For the same concentration of iron in solution, the bicarbonate alkalinity must exceed 46 ppm at pH 8 and 5 ppm at pH 9 to prevent the further solution of iron. Thus it may be seen that the solution of iron may be avoided, if the pH is raised above 8 or 9 and the water contains relatively high bicarbonate alkalinity. This points to two methods of corrective treatment which are in common use now, the use of soda ash to raise the pH value and the use of the calcium carbonate equilibrium.

The conventional explanation for the effectiveness of the calcium carbonate equilibrium for the protection of iron is that a protective coating of calcium carbonate is laid down on the surface of the metal, which prevents the access of water to the metal. This explanation is not a satisfactory one, because it is well known that calcium carbonate is too porous to prevent the penetration of water through it to the metal. A better explanation of the effectiveness of this treatment

process is that the presence of solid calcium carbonate requires the simultaneous presence of carbonate ion in solution. If the concentration of this carbonate ion is sufficient and the pH value in the pores of the calcium carbonate is high enough, protection is effective.

A study of reactions 36 and 74 indicates that, with waters relatively free of carbonate, if the pH value is raised to 9.5 or 10 and there is a slight amount of iron in solution, ferrous hydroxide will be formed at the anode and no further solution of iron will occur. This points to a third method of corrective treatment, namely the addition of lime or sodium hydroxide to raise the pH value to 9.5 or 10.

A fourth anodic reaction for iron in terms of phosphate is shown in Table 2. The data available to the author were not sufficient to compute the standard electrode potential of this reaction. Hence no analysis has been made of the conditions favorable for this form of anodic protection. This reaction is of interest in view of the use of phosphate compounds in corrective treatment. Sodium hexametaphosphate is widely used to prevent the formation of red water. Its action in this connection is as a peptizing agent, to prevent the growth of iron-rust crystals. They are thus kept so finely divided that they no longer impart color to the water. Some claims have been made that this chemical also prevents the solution of iron but these claims have not been well substantiated. It is doubtless true, from an examination of the phosphate reaction in Table 2, that protection against the solution of iron may be obtained by phosphate in the same manner as with carbonate and hydroxyl, but the effectiveness of the protection must also depend upon the pH value of the water.

Principal Cathodic Reactions. The principal cathodic reactions in the corrosion of iron in fresh water, both with and without chlorine, are shown in Table 2. The reactions at the cathode go from right to left, in a direction opposite to the direction of the arrows shown in Table 2. The products formed in the absence of chlorine are hydrogen gas, water, carbonate ion and water, or iron rust, depending upon which reaction prevails.

In the conventional literature on corrosion reaction 100, the plating out of hydrogen ions is the only cathodic reaction considered. In conventional theory it is assumed that, if the water contains no dissolved oxygen, hydrogen is liberated as hydrogen gas and, if the

water contains dissolved oxygen, the molecular hydrogen produced from reaction 100 is assumed to react with dissolved oxygen to produce water. It is difficult to conceive of a reaction between molecular hydrogen and molecular oxygen to produce water at ordinary temperatures. The production of water at the cathode is best explained by reaction 198.

A comparative study of these reactions indicates that reaction 100 cannot take place at ordinary pH values, unless the water is devoid of dissolved oxygen. Reaction 100 is of significance only if the water is very acid—pH value 2 or less—and contains little or no dissolved oxygen. Under these conditions, the voltage required to make reaction 100 go must exceed the single-electrode potential by an amount in excess of the "hydrogen over-voltage" and the hydrogen gas must be released as fast as it is formed. Hence, contrary to popular belief, reaction 100 is not of importance in ordinary corrosion processes. The reaction has been shown in Table 2 only because it is given such prominence in conventional corrosion theory.

All the other cathodic reactions which are of significance, in the absence of chlorine, require dissolved oxygen. The rate at which any of these reactions goes to the left is directly proportional to the concentration of dissolved oxygen in the water. This is well known and the removal of dissolved oxygen from the water is a well-known method for the treatment of industrial waters to retard corrosion. This method of treatment is relatively expensive and it is not practicable for municipal supplies, because it would require the elimination of open-surface reservoirs in the distribution system.

A comparative study of reactions 134 and 144 indicates that they are equally cathodic, because of the equilibrium which will be maintained between the three forms of carbonate. A comparative study of reactions 134, 198 and 220 indicates that reaction 220 is the most cathodic in the presence of a trace of dissolved iron and will take place at all cathodic areas where there is any dissolved iron. In the absence of dissolved iron at the cathode and in the presence of any carbonic acid or bicarbonate, both water and carbonate will be formed at the cathode. Since nearly all natural waters contain bicarbonate alkalinity or carbonic acid, reactions 134, 144 and 198 are equally cathodic and will take place at the cathode, if there is no dissolved iron present.

It will be noted that reaction 220 is the only one of the above-discussed cathodic reactions which produces a solid coating. This coating, ordinary iron rust, might be considered as a protective coating, in the sense that corrosion will stop when there is sufficient rust on the metal to insulate the surface against the flow of electrons through the coating. Ferric iron rust is known, however, to build up in relatively thick deposits without apparent effect upon the corrosion process. Moreover, since the production of ferric iron rust is one of the causes of red water and since rust on the pipe walls reduces the hydraulic capacity of the pipe, one of the primary aims of protection against corrosion is to avoid the formation of ferric iron rust.

The effect of chlorine on the corrosion of iron is indicated by the last three reactions in Table 2. The cathodic reactions with chlorine are similar to those with oxygen, chlorine replacing oxygen as the oxidizing agent. An examination of reactions 158 and 233 indicates that they are equally cathodic, because of the equilibrium which exists between hypochlorite and chlorite ion. A comparative study of reactions 233 and 240 indicates that, with a trace of iron in solution, reaction 240 is cathodic to reaction 233. A comparison of reactions 220 and 233 indicates that, with a measurable quantity of chlorine in the water, reaction 233 is cathodic, unless the chloride content of the water is very high. This comparison indicates that chlorine is a stronger oxidizing agent than oxygen and that, when present in solution, chlorine will increase the overall potential causing corrosion.

Corrective Treatment. From the above discussion it is evident that corrective treatment of the water for protection against corrosion of iron may be aimed at retardation of either the anodic reactions or the cathodic reactions, or both. Protection at the anode is the more practical method for municipal water supplies, inasmuch as it retards the solution of iron. Since most of our supplies are chlorinated, protection at the cathode will be ineffective.

Corrosion of Lead in Fresh Water

The principal reactions which take place in the corrosion of lead in fresh water are shown in Table 3. The process is the same as has been described for iron. There are, however, some differences in the products formed and these differences must be examined, to determine

TABLE 3.—CORROSION OF LEAD IN FRESH WATER

		E°_{25}, volts
Principal Anodic Reactions		
62.	$2\ OH^- + \underline{Pb} \longrightarrow \underline{Pb(OH)_2} + 2e^-$	+0.585
67.	$3\ OH^- + \underline{Pb} \longrightarrow \underline{HPbO_2^-} + H_2O + 2e^-$	+0.54
69.	$CO_3^{--} + \underline{Pb} \longrightarrow \underline{PbCO_3} + 2e^-$	+0.506
72a.	$HPO_4^{--} + \underline{Pb} \longrightarrow \underline{PbHPO_4} + 2e^-$	+0.463
92.	$\underline{Pb} \longrightarrow Pb^{++} + 2e^-$	+0.126
Principal Cathodic Reactions		
100.	$H_{2_{gas}} \longrightarrow 2H^+ + 2e^-$	+0.000
134.	$2\ CO_3^{--} + H_2O \longrightarrow 2\ HCO_3^- + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.66
144.	$CO_3^{--} + H_2O \longrightarrow H_2CO_3 + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.779
153.	$HPbO_2^- \longrightarrow O_{2_{aq}} + Pb^{++} + H^+ + 2e^-$	—0.857
180.	$\underline{PbO_2} \longrightarrow O_{2_{aq}} + Pb^{++} + 2e^-$	—1.086
198.	$H_2O \longrightarrow \frac{1}{2}\ O_{2_{aq}} + 2H^+ + 2e^-$	—1.273
Principal Cathodic Reactions in Chlorinated Water		
158.	$Cl^- + 2\ OH^- \longrightarrow OCl^- + H_2O + 2e^-$	—0.878
233.	$Cl^- + H_2O \longrightarrow HOCl + H^+ + 2e^-$	—1.49
237.	$2\ Cl^- + 2\ H^+ + \underline{PbO_2} \longrightarrow 2\ HOCl + Pb^{++} + 2e^-$	—1.53

whether corrective treatment for protection of iron will have an adverse effect upon the protection of lead.

Principal Anodic Reactions. A comparative study of reactions 67 and 92 indicates that these two reactions are equally anodic, because of the equilibrium which is maintained in solution between Pb^{++} and $HPbO_2^-$. The ratio between these ions depends only on the pH value. A comparative study of reactions 62 and 67 indicates that 67 is anodic to 62 under all ordinary conditions. Hence, anodic protection of lead by means of hydroxyl is not effective, as is the case with iron, and an increase in pH value of itself gives no protection.

A comparison of reactions 69 and 92 indicates that, with no more than 0.1 ppm of lead in solution, 69 is anodic to 92, when the carbonate and bicarbonate alkalinity exceeds 63 ppm at pH 7 and exceeds about 7 ppm at pH 8 to 10. It is evident, therefore, that protection against the solution of lead may be obtained by the usual amounts

of carbonate and bicarbonate alkalinity present in New England waters in the pH range 8 to 10; and when the carbonates are inadequate, corrective treatment with soda ash is effective. The carbonate alkalinity is not very effective at pH values much above 10.

An examination of reactions 72a and 92 indicates that 72a is anodic to 92 with less concentration of phosphate than is required for similar protection with carbonate. Phosphate ion is of little importance as a normal constituent of water but may be added for corrective treatment.

Principal Cathodic Reactions. The cathodic reactions with lead are similar to those in the corrosion of iron, except that a soluble lead oxide may be formed under certain conditions. A comparative study of the cathodic reactions in Table 3 indicates that, with a normal concentration of dissolved lead—0.1 ppm or less—solid lead oxide, PbO_2, is formed at the cathode at pH values above 8. At pH values of about 6.5 to 8, either water or water and carbonate are formed at the cathode. At pH values below about 6.5, biplumbate ion is formed at the cathode.

With chlorinated water the cathodic reactions shown at the bottom of Table 3 will take place, instead of those involving dissolved oxygen. With a trace or more of dissolved lead reaction 237 is the most cathodic.

Corrective Treatment. It appears from the above discussion that corrective treatment for anodic protection of lead requires a pH range between 8 and 10 and an adequate carbonate and bicarbonate alkalinity. A rise in the pH value is of itself ineffective, in that it may cause the solution of lead at the anode as biplumbate ion.

Corrosion of Copper in Fresh Water

The principal reactions in the corrosion of copper in fresh water are shown in Table 4. It will be noted in the table that ammonia has a deleterious effect in the corrosion of copper. It will also be noted that the voltage for the anodic reactions is much lower than is the case for either iron or lead. Because of this fact reaction 100, the plating-out of hydrogen ions at the cathode, does not take place in the corrosion of copper. Also because of this fact the rate of corrosion of copper is much less than for the other metals.

TABLE 4.—CORROSION OF COPPER IN FRESH WATER

		E°_{25}, volts
Principal Anodic Reactions		
80.	$2\ OH^- + 2\ \underline{Cu} \longrightarrow \underline{Cu_2O} + H_2O + 2e^-$	+0.361
96.	$2\ NH_{3_{aq}} + \underline{Cu} \longrightarrow \underline{Cu(NH_3)_2}^+ + e^-$	+0.11
98.	$4\ NH_{3_{aq}} + \underline{Cu} \longrightarrow Cu(NH_3)_4^{++} + 2e^-$	+0.05
116.	$\underline{Cu} \longrightarrow Cu^{++} + 2e^-$	—0.345
124.	$\underline{Cu} \longrightarrow Cu^+ + e^-$	—0.552
Principal Cathodic Reactions		
134.	$2\ CO_3^{--} + H_2O \longrightarrow 2\ HCO_3^- + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.66
144.	$CO_3^{--} + H_2O \longrightarrow H_2CO_3 + \frac{1}{2}\ O_{2_{aq}} + 2e^-$	—0.779
174.	$\underline{Cu(OH)_2} \longrightarrow \frac{1}{2}\ O_{2_{aq}} + Cu^{++} + H_2O + 2e^-$	—1.01
198.	$H_2O \longrightarrow \frac{1}{2}\ O_{2_{aq}} + 2H^+ + 2e^-$	—1.273
Principal Cathodic Reactions in Chlorinated Water		
158.	$Cl^- + 2\ OH^- \longrightarrow OCl^- + H_2O + 2e^-$	—0.878
218.	$2\ Cl^- + \underline{Cu(OH)_2} \longrightarrow 2\ HOCl + Cu^{++} + 4e^-$	—1.365
233.	$Cl^- + H_2O \longrightarrow HOCl + H^+ + 2e^-$	—1.49

Principal Anodic Reactions. A comparative study of reactions 116 and 124 indicates that both cuprous and cupric ions may be formed in the corrosion of copper, the concentration being about equal when there is 0.01 ppm of cupric copper in solution. With greater amounts in solution the ratio of cupric to cuprous copper increases rapidly. Hence reaction 116 is the more important of the two.

A comparative study of reactions 80 and 116 indicates that, with traces of copper in solution, 80 is anodic to 116 at pH values above 5.5. Therefore copper is protected from solution at pH values above 5.5 by the formation of cuprous oxide at the anode. If, however, there is an appreciable amount of ammonia present in solution, this protection may be ineffective. A comparative study of reactions 80 and 98 indicates that, with 17 ppm of ammonia in solution, 80 is anodic to 98 above pH 9 and cathodic below pH 8. If the ammonia concentration is 1.7 ppm, 80 is anodic to 98 above pH 8 and cathodic below pH 6. It is evident, therefore, that the use of ammonia with chlorination will cause the corrosion of copper, unless the pH value is above 8 or 9.

It will be noted from Table 4 that no anodic reaction involving carbonate has been listed. A study of the reaction indicates that anodic protection of copper by carbonate is ineffective. The presence of carbonate ions, however, does not interfere with the anodic protection of copper by hydroxyl.

Principal Cathodic Reactions. The principal cathodic reactions in the corrosion of copper are similar to those for iron, except that reaction 100 does not take place. A comparative study of reactions 174 and 198 indicates that 174 takes place above pH 7 and 198 takes place at pH values below about 7. Since reactions 134, 144 and 198 are equally cathodic, it follows that below pH 7 water and carbonate are formed at the cathode and above pH 7 cupric hydroxide is formed at the cathode.

The cathodic reactions in chlorinated water are similar to those with iron and take precedence over the corresponding reactions with dissolved oxygen.

Corrective Treatment. It is evident from the above discussion that corrective treatment is not required for the protection of copper under ordinary conditions, if the pH value is above 7, but that if ammonia is present the pH must be raised above 8 or 9 to prevent the solution of copper at the anode.

Corrosion of Brass in Fresh Water

Brass is an alloy of copper and zinc, the relative amounts ranging from 60% copper and 40% zinc in yellow brass to more than 90% copper in red brass. As has been shown above, the copper is not very readily corrosive in natural waters. The corrosion of brass usually takes place by dezincification, or zinc solution, which appears to be greatest in soft waters of low pH value and with high zinc content in the brass. The principal anodic reactions which take place in the corrosion of zinc are shown in Table 5. The standard electrode potentials of all the anodic reactions for zinc corrosion are much higher than those for copper. Hence, in brass the anodic areas are on the zinc and the cathodic areas are on the copper. Therefore no cathodic reactions are shown in Table 5.

Principal Anodic Reactions. A comparative study of reactions 18 and 22 indicates that reaction 18 is anodic above pH 6 to 7 and

TABLE 5.—CORROSION OF ZINC IN FRESH WATER

		E°_{25}, volts
Principal Anodic Reactions		
18.	$2\ OH^- + \underline{Zn} \longrightarrow \underline{Zn(OH)_2} + 2e^-$	+1.42
22.	$4\ OH^- + \underline{Zn} \longrightarrow \underline{ZnO_2^{--}} + 2\ H_2O + 2e^-$	+1.216
28.	$4\ NH_{3_{aq}} + \underline{Zn} \longrightarrow Zn(NH_3)_4^{++} + 2e^-$	+1.03
49.	$\underline{Zn} \longrightarrow Zn^{++} + 2e^-$	+0.762

reaction 22 is anodic at lower pH values. A comparative study of reactions 22 and 49 indicates that they are equally anodic, because of the equilibrium which is maintained between the two forms of zinc ion. Reaction 28 is relatively unimportant, except for very high concentrations of ammonia. It appears, therefore, that at pH values below 6 to 7 zinc will dissolve into one or more soluble forms of zinc ion, and above pH 7 zinc is protected at the anode by the formation of zinc hydroxide. A study of the reaction involving carbonate indicates that zinc carbonate is not formed at the anode and carbonate is, therefore, no protection against the solution of zinc.

Corrective Treatment. Corrective treatment of water for the protection of brass against corrosion involves a raise in the pH value sufficient to produce zinc hydroxide at the anode. pH values above 7 are reasonably effective but the higher the pH value, the more effective is the treatment.

CONCLUSIONS

This paper and Part I, which comprises the basic theory, have been presented in the hope that they will throw some light on what actually takes place in the corrosion of metals and will lead to more effective treatment of water supplies for protection against corrosion. The methods of analysis to determine what reactions take place have been presented and applications of the theory to practical use have been made. The analyses have been based upon equilibrium conditions and the method gives no indication as to the absolute rate of corrosion. The method does, however, form a basis by which rates of corrosion may be measured experimentally. It is to be hoped that this presentation may serve as a basis for a program of experimental measurements on rates of corrosion.

The application of the methods for determining what reactions will take place in a particular case of corrosion requires that all conceivable reactions be examined. The author has used in this study, with the exception of reaction 72a, only reactions which are listed in Table 1 and he makes no claim that all conceivable reactions are included therein. No reactions, for example, with the negative ions of organic acids, except carbonic, are listed. If, therefore, others have found that corrosion proceeds in a different manner from that described in this paper, it may be explained in terms of half-cell reactions not considered by the author.

WATER SUPPLY DEVELOPMENTS IN BOSTON, MASSACHUSETTS

And Recent Additions to the Works†

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Metropolitan District Organization

Boston, the capital city of the State of Massachusetts, has been a pioneer in municipal regional development within an area which has come to be described as a metropolitan district. It is difficult to say with certainty whether the first incentive to the pooling of municipal interests in an organized district came first from the need of more adequate sewerage or of a better water supply.

The State Legislature, in 1889, created a Board of three Metropolitan Sewerage Commissioners to provide systems of sewage disposal for Boston and seventeen other municipalities. Later action by the Legislature, in 1897, resulted in the beginning of the Metropolitan Sewerage Systems. The number of municipalities has since been increased until the present two Metropolitan Sewerage Districts, the North and the South, include thirty-three municipalities. As another example of such district organization, one of the earliest co-operative municipal improvements and one which has given the Boston of today one of its most distinctive features, was the development in 1904, in the unsightly tidal estuary of the Charles River, of a constant-level fresh-water basin at approximately four-fifths of the height between low and high tide extending for about 8 miles through the heart of the district.

However, the first organization of the municipalities in this entire metropolitan area on a large scale was for the purpose of pooling their several interests to obtain a safe and adequate water supply. In 1893, the State Board of Health reported to the Legislature the

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results of a thorough investigation of the question of water supply for the City of Boston, and of its many suburbs within a radius of ten miles from the State House. The investigation was made under the direction of Frederick P. Stearns, Chief Engineer of the Board. As a result of this report, the Legislature in 1895 created the Metropolitan Water Board consisting of three Commissioners each holding office for a 5-year term. The Act created a Metropolitan Water District, which within a few years of its inception included Boston and seventeen other cities and towns. Another municipality joined in 1909 and another in 1925, making the present total of twenty municipalities.

The Water Board continued the services of Mr. Stearns as Chief Engineer, to develop the new supply which the State Board of Health had recommended. This involved principally: the taking over of certain municipal pumping stations and distributing reservoirs, aqueducts, and also a system of storage reservoirs which the City of Boston had previously acquired and the largest of which it had under construction; the completion of the works already begun at the headwaters of the Sudbury River and the construction of a new reservoir on the Nashua River, the Wachusett Reservoir; the construction of the large grade-line Wachusett Aqueduct to divert the water from this new reservoir into the Sudbury area; and the construction of the grade-line Weston Aqueduct which, together with Boston's older grade-line aqueducts, was designed to bring the combined supply to the distribution system of the District.

In 1901 the water and sewerage boards were consolidated by the Legislature into the Metropolitan Water and Sewerage Board, of three commissioners, each serving five years at a time. In 1919 a still further consolidation was made by the Legislature to include also a Metropolitan Park Commission, which had been in existence as an independent group of municipalities for the previous 24 years. This consolidation created the Metropolitan District Commission as it exists today, with three Divisions, one maintaining and operating the Metropolitan Water Works, one the Metropolitan Sewerage Works, and one the Metropolitan Parks, the last including a police force for the park system.

The original water and sewerage boards were created mainly

for a job of construction, but they continued to maintain and operate the works after the construction was completed. At the present time the organizations in the three Divisions of the Metropolitan District Commission are particularly well adapted to the job of maintenance and operation and they only incidentally undertake any new construction.

Following this pioneering in the Metropolitan District field, many other large centres of population in the United States approached their problems in a similar way. Ordinarily such developments have had a distinctly municipal character on account of the preponderant interests of a single large city. For example, Hartford, the capital city of the State of Connecticut, organized in 1930 a Water Bureau of the Metropolitan District, consisting of the City of Hartford and five small surrounding towns. They constructed a system of reservoirs centering around the Napuag Reservoir, and since then have from time to time done a considerable amount of new construction and at the same time maintained and operated the entire system, retailing the water directly to the consumers instead of wholesaling it to the municipalities as in the case of the Boston Metropolitan Water District.

An outstanding example of the creation of a permanent engineering organization solely for the design and construction of large water supply works is that of the City of New York, in 1905. Its works have been maintained and operated by a separate municipal organization called the Department of Water Supply, Gas and Electricity. In 1905 it became apparent that the City must embark on an extensive development of new sources in order to meet growing needs, and for that purpose the Board of Water Supply of the City of New York was created. Unlike Boston's previous venture, this was a municipal organization, but, because New York is a political organization of many large boroughs, it has all the characteristics of a Metropolitan District of many municipalities.

The engineering organization for this construction board was first assembled under the leadership of J. Waldo Smith as Chief Engineer. From the start it has earned an enviable reputation in the design and construction of large municipal works. The City of New York found that in order to provide for the expansion of its

water supply needs it was necessary to keep this Board of Water Supply as a permanent organization entirely disassociated from problems of maintenance and operation, and engaged only in the design and construction of extensions to the water supply system.

The Metropolitan District Water Supply Commission

The New York Board undoubtedly furnished Massachusetts later the precedent which led to the creation by the Legislature in 1926 of the Metropolitan District Water Supply Commission. This Commission, of three members, was created as a special temporary commission for the construction of new works to extend the water supply sources of the District for the appropriated sum of $65,000,000. Only two of the commissioners are appointed by the Governor, every five years, the chairman being the chairman of the permanent commission for maintenance, the Metropolitan District Commission, acting ex-officio.

Three years after the construction commission was created a major business depression, which was world-wide, materially reduced the water demands so that the period of construction could economically be stretched over a longer period than originally contemplated. Furthermore, the entire job of extending the sources of supply was done for about $12,000,000 less than the appropriation, and as a result the Legislature, in 1938, instructed the commission to construct also a new pressure aqueduct which would not only give much needed additional capacity for getting the supply into the centres of actual consumption but also allow the discontinuance from regular use of some of the older storage reservoirs and aqueducts. These were subject to increasing pollution due to the encroachment of population on the watersheds and to leakage into the grade-line aqueducts. The war has interfered with the completion of about eight million dollars' worth of the aqueduct construction.

In the meantime, the Legislature in 1941 appropriated $15,000,000 for new sewerage projects as an initial step towards the improvement of conditions in Boston Harbor, which is an important recreational centre but which is polluted by the addition of 250 million gallons of sewage daily, and toward the improvement of the condition of the Charles River Basin which has been gradually deteriorating, due in no small part to the diversion from points upstream of water

for the domestic supply of several municipalities that have not yet joined the Metropolitan Water District. The design and construction of these sewerage projects was given by the Legislature to the Metropolitan District Water Supply Commission, since it was well organized for major construction work of this character. As a result, this commission, although nominally a temporary commission created to construct specific new works and turn them over to the permanent maintenance organization, has already been operating for about 18 years, is organized for large-scale design and construction work, and has a considerable construction program ahead of it at the present time.

Water Supply Works Prior to the Recent Additions

The water supply works of the district at the time the recent extensions were undertaken consisted principally of the following:

Wachusett Reservoir.—This originally impounded the run-off from 118.19 square miles at the head waters of the Nashua River, 32 miles from the State House. Its surface area is 6.5 square miles. The Wachusett Dam is of masonry on bedrock foundation with relatively little excavation, 146 feet high, 1,423 feet long; spillway 452 feet long. The capacity above the lowest intake sill, elev. 330 above Boston City Base or mean low tide, is 9.4 billion U. S. gallons, area 1,440 acres; at the masonry spillway crest, elev. 395, 65.0 billion gallons, 4,140 acres; at the maximum flow line with stop logs, elev. 396.5, 67.0 billion gallons; 4,190 acres.

The reservoir was first filled in 1908. Since that time the City of Worcester, having rights in the area although not a member of the District, has been allowed by the Legislature to divert 9.35 square miles for its water supply and to divert from the so-called Quinapoxet section not more than 10 mgd from an additional 20.7 square miles. Another 1.15 square miles has been diverted for sanitary protection, so that the net drainage area today is about 87.0 square miles, and in addition there are occasional flood flows from the Worcester areas.

The construction cost of the Wachusett Reservoir system has amounted to approximately $12,800,000.

Wachusett Aqueduct.—A grade-line cut-and-cover aqueduct, of horseshoe section, 11 ft. 6 in. wide x 10 ft. 6 in. high, diverts the water

from the tailrace of waterwheels at Wachusett Dam on the Nashua River, elev. 295, about 47,200 feet to an outlet chamber in the adjoining Sudbury River watershed, elev. 278.5. About 10,400 feet of the aqueduct is in tunnel at grade. Its capacity is 350 mgd.

The construction cost of the Wachusett Aqueduct has amounted to approximately $1,800,000.

Sudbury Reservoirs.—The Sudbury system originally included seven reservoirs. Five of these located on the South Branch of the Sudbury River are small, the available capacity of the largest being but little over one billion gallons, and have been discontinued from regular use for many years. The only two in regular use at the time the additions were started were on the North Branch, drainage area 27.68 sq. miles, and included the main Sudbury Reservoir: drainage area 22.28 square miles; area 1.292 acres at the flow line, elev. 260; available capacity 6 billion gallons; first filled in 1898.

The construction cost of the Sudbury Reservoir system, without depreciation, has amounted to approximately $7,655,000. This includes the original purchase from Boston for about $4,404,897.

Weston Aqueduct.—A grade-line cut-and-cover aqueduct, of horseshoe section 13 ft. 2 in. wide x 12 ft. 2 in. high, with a capacity of 300 mgd extends from the tailrace of a low-head waterwheel at Sudbury Dam, elev. 228, about 71,000 feet long to a connection with the low-service supply mains of the water district. Near this connection is the 200-million gallon Weston low-service Distributing Reservoir at elev. 200.

The construction cost of the Weston Aqueduct including the reservoir has amounted to approximately $7,030,000.

Sudbury Aqueduct.—A grade-line cut-and-cover aqueduct, of horseshoe section, 9 ft. wide x 7 ft. 8 in. high, with a capacity of 100 mgd extends from the tailrace of a high-head waterwheel at Sudbury Dam. This aqueduct, as originally used by the City of Boston, took the water from the lowest of the Sudbury reservoirs, elev. 169.32, about 83,800 feet to the Chestnut Hill Pumping Station in the District. To take water from the two reservoirs on the North Branch only, extensions were built by the District, consisting of two parallel 48 in. cast-iron pipes.

The Sudbury Aqueduct was purchased from the City of Boston for $3,704,515.

Distribution System.—The District's distribution system delivers water to Venturi meters generally located at the town lines of the various member municipalities.

Approximately one-half of the supply is delivered to the low service at elevations varying from about 140 to 190; about one-third is pumped at the Chestnut Hill Pumping Station to the Southern High Service at elevations varying mostly from about 210 to 250. About one-tenth is pumped at the Spot Pond Pumping Station to the Northern High Service at elevations varying mostly from 200 to 260. Five per cent or more of the total consumption is pumped to extra high standpipes beyond the reach of the regular distributing reservoirs.

Recent Additions West of Wachusett Reservoir

The new sources of supply from the Ware and Swift rivers, which the construction commission has recently added, complete a plan which was originally adopted when the District was first organized. At that time, several radically different plans were considered including the pumping and filtration of water from the Merrimack River, a large industrial stream only 25 miles north of Boston. Considered also was an aqueduct approximately 200 miles long from Lake Winnepesaukee, a large lake in the State of New Hampshire, adjoining Massachusetts on the north. This lake is one of the largest in eastern United States. Such plans were rejected in favor of the policy which was fixed at that time of taking upland waters from relatively clean watersheds in the State of Massachusetts, reaching progressively further west, first to the Nashua River, as above noted, but with a definite intention of extending still further west to the Ware and Swift rivers when the needs of the District should require. The need was first reported to the Legislature by the Department of Public Health in 1919. By the time effective legislative action was taken, late in 1926, the need was such that we were required first to develop an emergency supply from the old South Sudbury source which would increase the available yield until the first step in the new sources could be completed. The new works consist principally of the following:

Quabbin Reservoir.—This latest reservoir, with a tributary drain-

age area of 186.0 square miles, is in the valley of the Swift River, a tributary of the Connecticut River, 66 miles from the State House. Of the total yield from this area, 20 mgd is required to be released continuously to mills downstream. The reservoir is created by two large earth dams, and is 18 miles long with an area of 38.6 square miles, a storage volume of 415 billion U. S. gallons and a shore line of about 177 miles, including the shores of over 100 rocky and mountainous islands.

Sixteen miles of railroad in the flooded area and about 11 miles more above the flow line were discontinued. About 40 miles of new substitute hard-surface highways were constructed. About 2,500 people, living in 650 houses in the area occupied by the reservoir and on adjacent land, have been required to find other homes in the surrounding towns. However, due to the authority given to the Commission to acquire property by purchase as well as by eminent domain, it was possible to give ample time for these home-owners to readjust themselves. The District is most fortunate in securing such an ideal reservoir site in the most sparsely populated section of the State. Only about 21 persons per square mile resided in the entire watershed prior to the Commission's operations. About 80,000 acres have been acquired for sanitary protection and construction purposes.

Quabbin Park Cemetery, located just south of the reservoir in the town of Ware, was constructed for the reinterment of bodies buried in 34 cemeteries in the area which was flooded by the reservoir and in adjacent areas. Although the Commission gave the heirs of those buried in the Quabbin area the choice of having their kin reinterred in other localities, so pleased have they been with the fitting character of the new cemetery that practically all of the bodies removed have been buried in Quabbin Park Cemetery. This cemetery has been transformed from brushland into landscaped areas with access drives. The Commission has arranged for the removal of bodies under strict and careful supervision making very thorough investigations in each individual case, and keeping extensive records as to identity and location of burials.

The impounding of water in Quabbin Reservoir was begun in July, 1939 and over 315 billion gallons of water have been stored to date, and an area of about 37 square miles flooded. Four townships

were so completely inundated, or isolated, that their corporate existence was discontinued, involving many changes in both town and county boundaries. At the end of the last fiscal year, November 30, 1944, Quabbin Reservoir was completed, except some work of reforestation, watershed and protection. However, not all the diversion damages were settled.

At that time the district had spent for the reservoir, not including the dam and dike, $20,051,352. A large portion of this cost, $9,671,308, was for real estate. The average cost per acre of all the real estate settlements, including those in the Ware watershed as well as Quabbin, was $98.91 for the net settlement and $111.46 gross including all indirect real estate charges. This does not include any of the cemetery costs which totalled $279,928, including $122,842 for the work of removal and reinterment in Quabbin Park Cemetery at approximately $18.00 per interment. Some of the principal construction contracts were for:

Bituminous macadam highways (7 main contracts, 38.3 miles)	$1,602,634
Clearing (4 main contracts)	1,312,874

Winsor Dam and Quabbin Dike.—Two earth dams were built to impound the waters of the new reservoir. The main dam across the Swift River, named Winsor Dam over six years ago after the late Chief Engineer, Frank E. Winsor, is 2,640 feet long, 170 feet high above the bed of the river and 295 feet above the sound ledge foundation; it contains about 4,000,000 cubic yards of selected earth fill above the original surface. A 400-foot long spillway, with its crest at elev. 530 above mean low tide, was cut through a rock gorge east of the dam and fixes the high water elevation of the reservoir. This length includes a section depressed two feet to elevation 528 and controlled by stop logs, useful as a flood control measure on account of the reservoir's great size in proportion to its drainage area. The smaller of the two dams, Quabbin Dike, located about three miles east of Winsor Dam, is 2,140 feet long, 135 feet high above the bed of Beaver Brook, and 264 feet above the sound ledge foundation; it contains about 2,500,000 cubic yards above the original surface.

Both these dams were built with a concrete core wall foundation

to sound ledge consisting of a row of caissons across the valley. Each caisson was 45 ft. x 9 ft. in horizontal cross section. The dam required 40 and the dike 31 of these in depths up to 125 feet. In the process of sinking them, the ground water level was lowered about 100 feet by continuous pumping from several of the caisson chambers. The superimposed embankments were built by the full hydraulic-fill method, from suitable borrow areas on lands of the Commonwealth generally located below full reservoir flow line.

Up to November 30, 1944, the district had spent $5,875,594 for Winsor Dam, which is complete except for the installation of a small waterwheel; and it had spent $2,903,509 for Quabbin Dike. Some of the principal construction contracts were for:

Stream control works at the Dam	$ 544,611
Exploratory Caissons	162,839
Caisson core wall at the Dike	1,151,602
Caisson core wall at the Dam	1,606,630
Quabbin Dike embankment	1,402,752
Winsor Dam embankment	2,439,728

Quabbin Aqueduct and Ware River Intake.—Quabbin Aqueduct, which extends from the easterly shore of Quabbin Reservoir to the upper end of Wachusett Reservoir, has a total length of 24.61 miles, twice as long as the Simplon Tunnel under the Alps. It is of horseshoe section, 11 ft. 0 in. wide x 12 ft. 9 in. high, constructed as a grade-line aqueduct, but operated as a pressure aqueduct. That is, the normal ground water level is, under all conditions, higher than the hydraulic gradient, except for a very short length at each end where the section is constructed as a circular, grouted pressure tunnel. About 13 miles from the Wachusett end the tunnel passes under the Ware River in Barre, where provision is made at the Ware River Intake Works to divert into the tunnel all flood flows in excess of 85 million gallons daily. The drainage area of the Ware River at the Intake Works is 96.8 square miles of which area the Commission now owns about 21,000 acres for proper sanitary protection of the water to be diverted.

The Intake Works, elev. 656, are higher than either Quabbin Reservoir, elev. 530, or Wachusett Reservoir, elev. 395, so that water

FIG. 1.—WINSOR DAM. THE MAIN DAM OF QUABBIN RESERVOIR.

FIG. 2.—LOOKING DOWN THE SEAPLANE RAMP ON ONE OF THE SMALLER BRANCHES OF QUABBIN RESERVOIR.

FIG. 3.—SINKING THE CAISSON CORE WALL TO SOUND LEDGE IN THE FOUNDATIONS OF WINSOR DAM.

FIG. 4.—CONSTRUCTION OF WINSOR DAM BY THE FULL HYDRAULIC METHOD.

may be diverted simultaneously into both Wachusett and Quabbin Reservoirs. The drop to tunnel grade is about 260 feet. The gates at the Wachusett Outlet Works will ordinarily be closed when diverting Ware River water, so that this water can be diverted westerly for storage in Quabbin before drawing it back through the tunnel to Wachusett Reservoir.

Water is discharged into Quabbin Reservoir not at the extreme westerly end of the aqueduct but through a special outlet shaft into a branch of the reservoir, from which it must pass around the northerly end of a large island joined to the mainland by means of a baffle dam so that the water from the Ware River will be thoroughly mixed with the water in the reservoir and receive the benefit of long-time storage before it is drawn for use. Water is taken into the tunnel for discharge to Wachusett Reservoir through Intake Works at Shaft 12 at the extreme westerly end of Quabbin Aqueduct.

The tunnel was excavated through solid rock for its entire distance and was lined with concrete. Its capacity is as follows:

From Ware to Quabbin	967 mgd
And simultaneously to Wachusett	1,033 mgd
Total	2,000 mgd
From Ware to Quabbin only (none to Wachusett)	615 mgd
From Quabbin to Wachusett	642 mgd
From Quabbin to Wachusett at grade	396 mgd

Up to November 30, 1944, the district had spent $15,081,686 for the Ware supply and for that portion of the aqueduct between Wachusett Reservoir and the Ware River Intake Works, and $5,420,396 for the extension of the aqueduct from the Ware Intake to Quabbin Reservoir. The construction is completed except for the installation of a waterwheel at the Wachusett end of the aqueduct. Of this total, $2,644,758 was for real estate for the protection of the Ware watershed. Some of the principal construction contracts were for:

Shaft sinking and tunnel driving and lining (6 contracts)	$13,832,827
Ware River Intake Works (2 main contracts)	355,694
9½-mile trunk sewer	238,709

Safe Yield of the Sources of Supply

The safe yield and its increase from the old Wachusett source by the addition of the new Quabbin source is shown in the following tabulation:

Source	Drainage area in square miles		Safe yield in million gals. daily: Before Quabbin Reservoir was available — Depletion of Wachusett (and N. Sudbury) storage limited for safe quality to 45 bil. gals.	Before Quabbin Reservoir was available — Total depletion in an emergency 65 bil. gals.	Combined operation of Wachusett Reservoir and Quabbin Reservoir
Wachusett, including Worcester's Quinapoxet Area		107.69			
North Sudbury		27.68			
		135.37	119	126	133
South Sudbury:					
Tributary to N. Sudbury	10.61		9	9	3
Not Tributary to N. Sudbury	23.67		10	10	
		34.28			
Cochituate		17.4	8	8	
		187.05	146	153	
Ware		96.8	11	31	40
		283.8	157	184	
Quabbin:					
East Branch	52.0			10	
Additional	134.0			194	
		186.0			154
		470.0			330
Possible Future Millers		272			189

Note that the new development, which provides an aggregate total safe yield of 330 mgd, more than doubled the former maximum yield of 153 mgd; and multiplied by about 2¾ the former yield of 119 mgd from sources of good quality.

An outstanding feature of the 415-billion gallon Quabbin Reservoir is its great size in proportion to the total of its natural drainage

area and the area made tributary to it by diversion. The reservoir basin is shaped like an elongated bowl with relatively flat bottom and steep sides, and although the storage in the upper layers, even to a depth of as much as 25 or 30 feet, adds nothing to the safe yield, nevertheless this extra height was added to the dam and dike at relatively small cost.

The Commission decided that it was best to provide this extra storage for two reasons; first, to continue the policy of relying principally on long-term storage for quality, and second, to anticipate diversions from additional watersheds in the future, even though somewhat remote. The original proposal for the new additions did, in fact, contemplate the diversion of flood flows from another adjoining watershed of over 270 square miles, namely, that of the Millers River. This river is conveniently located to the north and could easily and cheaply be diverted by gravity into Quabbin Reservoir, in case the use of the reservoir should be expanded to provide for the needs of other districts throughout the state, or in case such a diversion should be planned as a flood preventive measure. These are the only reasons which justified the construction of so large a reservoir, and on that account the Millers River is included in the tabulation of safe yield although there is no present authorization of such an additional development.

The table also shows how the new sources were economically developed in stages. First, certain of the South Sudbury sources were re-developed, including pipe lines from four of the headwaters reservoirs by-passing the principal sources of pollution in the watershed, in such a way as to be immediately useful in an emergency, but only to a limited extent available for continued use. This work was completed in 1927. Next, the easterly half only of Quabbin Aqueduct tunnel was built, extending westerly from Wachusett Reservoir to tap the flood flows of the Ware River. This construction was sufficiently completed in January, 1931, to allow the first diversion of the next available flood flows to be made from the Ware River to Wachusett Reservoir, although much of the operating equipment of the Intake Works was, at that time, housed in temporary wooden enclosures. Next, the tunnel was extended to the Quabbin Intake on the East Branch of the Swift River, the work being done

behind a massive steel bulkhead in the tunnel, which prevented interruption by the Ware diversions. This construction was sufficiently completed late in 1934 to allow diversions to be made from the East Branch, without benefit of storage, if such diversions had been found necessary. Next, the reservoir was completed late in 1939 and, by the first of April, 1940, sufficient storage had accumulated so that drafts could be made to Wachusett Reservoir as required.

Consumption from Metropolitan Sources

The 20 cities and towns which are members of the Metropolitan Water District had a population by the 1940 census of 1,509,376. Eighteen of these towns take their entire supply from District sources and consumed in the year 1944 an average of 162,449,100 gallons per day. The other two members consume between 10 and 11 million gallons daily and in the year 1944 took from District sources an average of 863,800 gallons per day, which is only a small percentage of their total requirements. In addition an average of approximately 1.55 million gallons daily was used by non-members and taken from the distribution mains in the District proper. In comparing these consumption figures with the safe yield, consideration must be given to the fact that the District sells from its aqueduct system, outside the District proper, an average of nearly 4 million gallons daily and also that the City of Worcester is entitled to pump from its allotted portion of the Wachusett Watershed a yearly total averaging 10 million gallons daily. The following is a record of the increase in these consumption requirements since the organization of the District in 1895 to and including 1944:

	Population of 20 member municipalities	Gals. consumed per cap. daily after services were practically 100% metered	Million gallons daily supplied from district sources				
			To member municipalities	To non-members within the district	En route along the aqueduct	To City of Worcester	Total
1895	779,581						
1900	897,475						
1905	974,877		116.08		1.88		117.96
1910	1,098,048		112.09		0.87		112.96
1915	1,234,790		101.94		1.06	0.13	103.13
1920	1,290,651		127.27		1.38		128.65
1925	1,398,702	97.6	128.86	0.06	3.00	2.58	134.50
1930	1,498,253	97.5	137.16	0.37	3.19	2.39	143.11
1935	1,548,282	90.9	131.27	0.45	3.43		135.15
1940	1,511,667	98.7	139.48	0.57	3.46		143.51
1941		102.4	145.94	0.89	3.01	1.92	151.76
1942		105.4	151.51	0.61	3.38	9.36	164.86
1943		109.4	158.30	0.90	3.51	3.16	166.87
1944		111.9	163.31	1.47	3.33	6.59	174.70
1945*	1,558,287	111.8	164.23	1.33	3.31	0.82	169.69

*These figures have been added to the original paper by the author.

Apportionment of the Annual Costs

The cost of water assessed against the various member municipalities consists of amounts appropriated annually by the State Legislature to be spent by the Metropolitan District Commission for maintenance and operation of the District's works and for additions to the supply mains and other minor construction costs, and of amounts necessary to meet the principal and interest requirements of bonds that were issued to finance the construction of the new works by the Metropolitan District Water Supply Commission and of the old works by the Metropolitan District Commission. The State Treasurer annually apportions the aggregate total of these amounts to the members, one-third in proportion to their tax valuation and two-thirds in proportion to their water consumption, the consumption figures necessarily being taken from the previous year. These assessments are the wholesale costs to which each municipality adds its own fixed and current charges for distribution.

The State Legislature has just passed, in the 1945 session, an Act designed to correct a situation which, it is felt, has resulted from the financing of the new works with bond issues of too short a term.

These were 30-year serial bonds, the life of which averages 15 years. The result has been a very material increase in the cost of water, which has kept a number of outside municipalities from abandoning their local supplies and joining the District. The new Act limits the amount of bond requirements to be included in the assessment each year and requires the State Treasurer to issue new "water-use-development bonds" to make up the deficiency. The effect will be to substitute an immediate reduction in the cost per million gallons to $40 and to maintain this constant, in lieu of a gradual but certain drop in the cost from the present figures of about $80 to about one-third of this in 25 years.

The following table shows how the total District assessment and the cost per million gallons have varied:

	Maintenance and all other costs except capital expenditures	Credit resulting from standby charges assessed against non-members in previous year	Annual appropriations for construction by M.D.C.	Principal and interest requirements of bonds and misc. interest		Total assessment	Average per mil. gals. consumed in the year of assessment
				Construction of old works by M.D.C.	Construction of new works by M.D.W.S.C.		
1905	$308,860			$1,848,962		$2,157,822	$50.80
1910	380,242			1,917,546		2,297,788	56.10
1915	417,623			1,746,707		2,164,330	58.20
1920	746,853			1,771,431		2,518,284	54.00
1925	741,180			1,786,236		2,527,416	53.70
1930	892,908		$425,000	1,821,158	$821,193	3,960,259	79.00
1935	936,450		450,000	1,757,172	1,908,122	5,051,744	105.50
1940	986,385		250,000	976,527	2,573,033	4,785,945	93.90
1941	1,005,090		280,000	751,714	2,631,676	4,668,480	87.60
1942	1,108,494		200,000	514,179	2,659,872	4,482,545	81.10
1943	1,635,475	$72,868	161,000	386,521	2,649,618	4,759,746	82.30
1944	1,376,957	36,434	176,000	313,971	2,834,728	4,665,222	78.30
1945*	1,166,584	36,333	72,917	260,615	3,104,299	4,568,032	76.20

*1945 figures added to original paper by author.

New Pressure Aqueduct

The Commission's most recent project, namely, its new Pressure Aqueduct is also in many respects the most interesting. Its construction was authorized by the Legislature in 1938. No additional appropriation was necessary on account of the grant of Federal funds by

the Public Works Administration of a little over $7,000,000 to cover 45 per cent of the construction cost. The authorization of the project followed an extensive joint report of the Commission and the Department of Public Health which pointed out the inadequacy of the old grade-line aqueducts to get the new supply in the District in a dependable and satisfactory manner. The old Cochituate and Sudbury Aqueducts, constructed by the City of Boston in 1848 and 1878, respectively, were particularly unsatisfactory on account of increasing leakage of pollution from the outside into the aqueducts. The grade-line Wachusett and Weston Aqueducts constructed by the District in 1898 and 1903, respectively, are maintained in good condition and, except for the fact that delivery is to the low service only, are of ample capacity. The new Pressure Aqueduct parallels and supplements the old Weston Aqueduct and delivery is to the grade of the high service. Hence, when it is completed after the war by the City Tunnel extension, it will eliminate the necessity for pumping to the southern high service.

The Pressure Aqueduct starts at the east terminal chamber of the Wachusett Aqueduct where a small protected forebay can serve

FIG. 5.—EXCAVATING THE 24.6-MILE QUABBIN AQUEDUCT TUNNEL.

as an equalizing basin between the grade-line and the Pressure Aqueduct. The Pressure Aqueduct then extends in 12 ft. 6 in. diameter cut-and-cover construction for a distance of two miles to a downtake shaft, then in 14 ft. tunnel under Sudbury Reservoir for three miles, then in 11 ft. 6 in. diameter cut-and-cover construction for a distance of 12½ miles to a downtake shaft just west of the Charles River and outside the thickly populated water district.

From this point the proposed so-called City Tunnel will extend to a connection with the southern high service at Chestnut Hill. Near the temporary eastern end at the downtake shaft at the Charles River, a connection consisting of a little over 2,000 feet of 7 ft. diameter cut-and-cover aqueduct connects with the existing Weston Aqueduct supply mains near the Weston Aqueduct terminal chamber; and by this means the increased head is made available on some of the present mains.

The Pressure Aqueduct not only can eliminate the regular use of the present grade-line aqueducts, but also by-passes Sudbury Reservoir with its populated shores, passing this reservoir at a hydraulic grade 15 feet above the reservoir flow line. The grade at the Wachusett Aqueduct terminal equalizing basin, where the Pressure Aqueduct will take the water, is elev. 278.5, above mean low tide. The gradient which limits the high points of the Pressure Aqueduct has been established so as to permit the aqueduct to be used in an emergency to draw directly from Sudbury Reservoir. In such an emergency, the grade in the future distribution centres will be lowered about 15 feet, and existing reservoirs on the Southern High Service at this lower grade will again function, but it will not again be necessary to resort to pumping.

The present construction includes a new high-level distributing reservoir which will ride on the aqueduct at elev. 274.5. This is matched by a similar reservoir a few feet lower, elev. 271, northeasterly of the district, on the Northern High Service, which, it is expected, will some day be connected in such a way as to eliminate pumping for that service also.

The cut-and-cover portions of the line aggregating 15 miles in length and also the short 7 ft. diameter connection to the supply mains were constructed of precast reinforced-concrete steel-cylinder

pipe. The 12½ ft. diameter sections were cast in 12 ft. lengths and the 11½ ft. diameter sections in 16 ft. lengths, the individual sections weighing up to 50 short tons apiece. In general a right-of-way for the construction of the Pressure Aqueduct 175 feet wide was taken, 50 feet of this being temporary easement for construction purposes only. A contract was awarded for the precast pipe on December 28, 1938, and the erection of an extensive plant for its manufacture in Natick was begun the following March. This plant turned out daily twenty-four complete 11½ ft. diameter units 16 feet long and five steel-cylinder and reinforcement-cage units for the 12½ diameter pipe. These latter large units were to be laid several miles from the Natick plant. Therefore a plant near the west end in Southborough was built so that the concrete could be poured there, and the cages were transported over the highways to the Southborough plant where the length of haul of the completed pipe units to the trench was very short. The entire 15 miles of pipe was manufactured in a single season, the contract being practically completed in December, 1939.

Some of the details of the construction of this aqueduct are sufficiently interesting to warrant detailed comment. In the precast units of reinforced concrete were embedded a steel cylinder, for

FIG. 6.—LAYING THE 138-INCH PIPE FOR THE PRESSURE AQUEDUCT BY GANTRY CRANE.

absolute watertightness under pressure, and two cages of reinforcing steel. On the inside of the pipe, two inches of unreinforced concrete cover the steel cylinder. Three major factors were taken into consideration in the design of this pipe, the internal working pressure, external loads on the empty structure and the dead load of the pipe itself. The last two factors influenced the stiffness required and affected the choice of wall thickness, which is 12 in. for the 150 in., 11 in. for the 138 in. and 8 in. for the 84 in. pipe.

The internal pressures allowed for varied from 100 to 220 feet head of water, no pipe being designed for less than a 100 feet.

The steel cylinders were designed to take 40 to 50 per cent of the stress induced by internal pressure, the remainder being carried by the reinforcing cages. The allowable stresses were purposely set low in order to prevent excessive cracking of the concrete under pressure and to ensure watertightness of the aqueduct even under some unexpected condition of deterioration of the concrete, or cages, and accident to the concrete envelope. The pipe is so stiff and strong that not only could it carry the load of earth refilling but highways could be permitted along or across the aqueduct without requiring an additional structural envelope. As the pipe has very strong walls, there was no necessity for designing it against collapse under vacuum.

The pipe joints were made of steel joint rings into which could be caulked a lead gasket for watertightness. The general features of the joint required that when the pipes were laid they would be self-centering without the aid of the gasket, that non-uniform settlement of adjoining pipes would not increase the bearing load on the gasket. To this end the rings were accurately formed so that, when adjoining pipes were put together a satisfactory sliding fit for the self-centering surfaces with a tolerance of not more than 1/32 in. from a close sliding fit was obtained. The gasket is a fibre-filled, extruded lead gasket of wedge shape, and made endless of proper circumference to fit loosely between the spigot ring and the bell ring of the joint. Reinforcing mesh was cast in the spigot end of each pipe to bond the mortar filling of the caulking recess to the pipe spigot, thus ensuring proper backing of the lead gasket at all times.

Compacted refill was placed around the pipe in addition to the construction of a cheap lean-concrete cradle that was poured around

the lower portion of the pipe for a width approximately equal to the internal diameter of the conduit. The purpose of this construction was materially to strengthen the resistance of the pipe walls to external loadings by creating a wide bearing across the base and compacted refill for support of the sides. The economical depth of laying the pipe was usually about 10 ft. in earth. Where the ground water level was more than 7 ft. above the invert of the pipe, it was necessary to weight the line against floating when empty and uncovered. This was done by increasing the size of the cradles and providing for anchoring the pipe to the cradle.

In order that the pipes could be manufactured by one contractor and laid by another in a predetermined location, the position of each joint along the line of aqueduct was predetermined, and pipes containing connections or forming curves could be cast of proper dimensions in advance. All curves were made with bevel-ended pipe, the angle of the bevel being about 4½ degrees. Plate-steel bulkheads were used for testing complete sections of the aqueduct, and the leakage requirement of less than 75 gals. per day per inch of diameter per mile of pipe was easily met.

FIG. 7.—ROLLING THE 150-INCH PIPE INTO THE PRESSURE AQUEDUCT TRENCH.

FIG. 8.—LAYING THE 150-INCH PIPE FOR THE PRESSURE AQUEDUCT BY LOCOMOTIVE CRANE.

FIG. 9.—THE FINISHED PRESSURE AQUEDUCT IS SHOWN TO GOVERNOR SALTONSTALL BY THE CHIEF ENGINEER.

The pipe was laid by three contractors and progress in the different sections varied from about 7 to 17 pipe sections per day on the average, with a maximum in one day varying from 20 to 32 sections. One contractor had a section which was on a long uniformly flat grade and he rolled the pipe into the trench on ramps located at several points and handled the sections in the trench with a locomotive crane on rails on the bottom of the trench. Another contractor used a gantry crane spanning the trench, and the third contractor used two caterpillar-tread cranes operating from one side of the trench, each crane handling one end of the pipe section. Several methods were used to lay pipe on steep grades. At the Sudbury River Crossing, the pipe sections were slid downhill into position over two rails laid to proper line and grade in the trench. The rails were greased, and two steel channel sections were placed between the concrete pipe and the rail to aid sliding. One contractor successfully laid pipe downhill on a steep grade with a gantry crane by anchoring each pipe on the slope from the inside by means of a cable fastened to the bell of the pipe being laid and carried back to a dead man erected in the aqueduct several hundred feet back of the top of the hill. The work as a whole was expeditiously done, even though two of the contractors had had no experience on similar work.

The cost of the cut-and-cover Pressure Aqueduct may be summarized briefly as follows: 9,680 feet of the 150-inch pipe cost about $60.00 per foot to manufacture and about $50.00 per foot to lay; 66,063 feet of the 138-inch pipe cost about $55.00 per foot to manufacture and about $30.00 to lay; 2,425 feet of the 84-inch pipe cost about $28.00 per foot to manufacture and about $15.00 per foot to lay. Up to Nov. 30, 1944, the District had spent $7,497,198 for the Pressure Aqueduct. In addition a large portion, $5,704,000 of the Federal P.W.A. grant was contributed to the aqueduct construction. The City Tunnel extension to Chestnut Hill remains to be completed. The principal construction contracts were for:

15,820-foot tunnel under Sudbury Reservoir	$2,933,832
Manufacturing precast pipe	4,198,409
Laying precast pipe (5 contracts)	2,707,523
Norumbega Distributing Reservoir	574,184
Shaft sinking for Intake to City Tunnel (2 contracts)	395,591

Author's Concluding Discussion*

Answering inquiries about the caissons sunk to sound ledge at Winsor Dam and Quabbin Dike and the reason for their use, this method of constructing the core wall for each of these dams was adopted because it was considered to be the most economical and satisfactory method of securing a tight foundation in the glacial debris of sand and gravel, which filled these gorges.

One alternative suggested is to make an open excavation, the side slopes of which would probably have to be as flat as 1:2½ at the bottom. The excavation would have to be protected by adequate cofferdams, the tops of which would have to be approximately 160 feet above the bottom of the excavation, perhaps more at the Swift River site. This method was used by the City of Providence, Rhode Island, in constructing its Scituate Dam in 1921. There the rock lay only about 80 feet below the river bed and the natural water level, instead of about 130 feet in our case. The side slopes of the excavation were 1:2 at the bottom. In order to secure a satisfactory cut-off, the ledge had to be exposed for a width of 60 feet; and a width of 30 feet in the center was sealed with a concrete blanket. The refilling contained an impervious rolled soil core, 77 feet wide. About 300,000 cubic yards were excavated and refilled, this portion of the work running well into the third year.

Another alternative suggested is to excavate in a sheeted trench. The author is satisfied that this can be done satisfactorily at great depth only where the material is practically free from boulders; and even in such case, it is difficult to make a satisfactorily tight seal into sound rock at the bottom. The difficulty is principally in the handling of water encountered. This method was used by the North Jersey Water District in constructing their dam at Wanaque, New Jersey, in 1920. The lowest point of the bedrock was about 85 feet or more below the river and the water table, and about 80 feet below the bottom of an excavated trench, from which the steel sheet piling was driven. A concrete core wall 12 feet thick was constructed in the sheeted trench.

An outstanding example of a situation which called unquestion-

*Due to limitations of space, it has been found impracticable to reprint all of the voluminous discussion, in which many British engineers participated.

ably for the caisson method to obtain a tight seal to bedrock, as well as a tight wall, is that of the Wyman Dam, built by the Central Maine Power Company in 1929 in Bingham, Maine. Here the bedrock was 120 feet below the Kennebec River and much of the material could only be described as a mass of boulders, allowing free passage of water.

Another alternative suggested is to tighten up the existing material by cement grout, asphalt or other injections. Such methods have been used to some extent in this country. The author has but little familiarity with them. They were not considered to be a satisfactory solution of the foundation problem at Quabbin.

Prior to advertising contracts for the caisson core walls, a contract was let for sinking two exploratory caissons at each site, which demonstrated the practicability of the scheme, enabled an early start to be made in lowering the ground-water level, and took much of the risk out of subsequent bids for the main job.

Answering further inquiries about details of the design and construction of the caissons, they were spaced so as to leave an end clearance of 18 inches. The ends were squared for a height of 7 feet above the cutting edge and above this an 18-inch recess was cast in each end. The resulting working space, 4½ feet wide, for excavating and filling the end joints proved ample, in spite of the fact that it was impossible to sink the caissons in precise vertical alignment. When the joint excavation reached the ground-water level, the excavated portion was filled with a concrete cap, keyed to the caissons, containing a 3-foot circular well, at the top of which an air lock, especially designed for the close quarters, was installed to permit continuing the excavation. Each caisson was left projecting at least 25 feet above the bottom of the open-cut trench or, where necessary, was extended to that height after seating. Hence the distance of travel in possible seepage along the caisson sides and across the top was about 60 feet. At the ends of the dam, the rock was uncovered and cleaned of porous material for a width varying from 60 feet at the bottom to 40 feet at the top. Over the central 30 feet, a blanket of concrete, in blocks of irregular shape to fit the contours, was cast, to insure contact with the impervious core material subsequently placed, and to eliminate the danger of piping, along and above cracks in the bedrock.

Answering inquiries about the type of embankment construction, the hydraulic-fill method should come as near to giving a stable, tight dam as any other, provided the right material is avaliable in the borrow pits, and it generally has the advantage of cheapness. The material available at Quabbin was ideal, in that satisfactorily coarse shoulders were obtained from the coarser ingredients and a satisfactorily tight core could be secured without danger from an excess of extreme fines. In fact, there was practically no true clay in the borrow pits. It is true that many of the dams constructed by this method in the United States have failed, but in all important cases, such failure has occurred during construction. This has generally been due to the fact that the only available material was unnecessarily fine, containing too much clay, and could not consolidate with sufficient rapidity. It is a truism that if such a dam does not fail during construction, it is perfectly safe thereafter. Changes in the water content of superfine and clay cores occur at an extremely slow rate and there are undoubtedly many such cores which can still be classed as fluid many years after construction. In any event, such a core could not dry out between periods of high water to such an extent as to permit dangerous leakage.

Answering inquiries about the consumption per capita, it is true that, compared with some of the large cities in the United States, this is a low rate of consumption, even though it appears high compared with the records in England. Fundamentally, consumptions here are high simply because the water is available and relatively cheap, and many industries use water from the domestic system which would not do so under conditions of scarcity. Also, there is quite a bit of waste, especially in the cities which report high rates of consumption, and the services in such cases are generally not metered. The data from the Boston Metropolitan District indicate that it would be reasonable to conclude that metering of the services in about a 10-year period reduced the consumption to about ¾ of what it might otherwise have been.

Answering inquiries about the rainfall and estimates of yield, the average rainfall recorded on the Swift River watershed during the last 26 years at 9 stations above Winsor Dam is 45.89 inches, and on the Ware River watershed during the last 22 years at 5 sta-

tions above the intake is 43.20 inches. In estimated yields, reliance is placed on records of the actual rate of flow, either on the stream in question or on an adjoining or nearby one. The United States Geological Survey has maintained for many years gaging stations on all important streams in the United States. Rainfall records even of short duration are very useful in comparing one watershed with another. The average run-off from the Swift River during the 27 years prior to the commencement of filling Quabbin Reservoir was 1.68 cu. ft. per sec. per sq. mi. Excellent records have been kept for the Nashua River above the District's Wachusett Dam since 1908, monthly comparisons made between rainfall and runoff, and estimates made of the yield per square mile of land area and per square mile of water area, in the manner outlined in a report of the New England Water Works Association's Committee on Yield of Drainage Areas, published in Vol. XXVIII, No. 4, of the Journal in 1915.

Answering inquiries as to the criteria for spillway design, although many important spillways in the United States have been designed on the basis of a factor of safety of something like two or more, applied to the maximum flood of record, the author believes that the best practice in the case of large and important works such as these in question is to determine from hydrologic studies what is the maximum concentration of run-off, both as to rate and duration, that can be considered a physical possibility, regardless of the past record on the stream, and not to give too much weight to the estimated probable frequency of its occurrence; and then to apply these data to the reservoir area and other physical characteristics. Many spillways have been built at the outlet of drainage areas about this size, on the basis of their ability to discharge in the neighborhood of 200 cu. ft. per sec. per sq. mi. A rate of 500 c.f.s. per sq. mi. into such a reservoir would be considered conservative. The duration of the flood or shape of the hydrograph is generally of more consequence than the peak rate. The author undoubtedly leans toward the conservative side in considering further that any spillway which has a constricted get-away channel, such as the deep, excavated rock gorge at Quabbin, necessitates a consideration of the possibility of clogging. The possibility is not at all unreasonable of a peculiar combination of severe winter conditions, which could cause a very

serious throttling by accumulations of snow and ice. A condition of this sort is well taken care of at Quabbin by providing a sort of depression in the rim of the reservoir, in the form of an earth cut with 100-foot bottom width and 6 feet higher than the main spillway crest. From the bottom of this channel an arched wall of concrete with a granite coping has been sunk to the ledge to form a low, arched dam 240 feet long, which, if ever overtopped, as is extremely improbable, would prevent erosion cutting back to the reservoir and would confine the discharge to a natural brook bed emptying into the river some distance below the dam.

Answering inquiries about the Ware River Intake Shaft, the control valves discharge through the shaft cap tangentially against the sidewalls of the shaft at an angle of about 30° with the horizontal. Projecting ribs on the cast-iron plates with which the shaft is lined form helical vanes, like a 6-pitch screw thread, which maintain the rotary motion of the falling water. Thus the water is pressed firmly by centrifugal force against the shaft wall and is given no opportunity to impinge directly against any surface. As a result, the energy is dissipated in a thin conical jet at the bottom. It is important to note, however, that the water dropped the entire distance only when the discharge was always easterly to the Wachusett Reservoir, before Quabbin Reservoir was completed, but that in the present ordinary use discharging westerly to Quabbin Reservoir the bottom 100 feet or more of the shaft is filled with water; also that increases in the rate of diversion lift the hydraulic gradient in the shaft until the capacity of the works is reached, when there is no residual head to lose in the drop into the shaft. Under no conditions, even with Quabbin shut off and the Wachusett gates wide open, could the drop be for the full depth of the shaft, unless the rate of discharge is less than about 450 c.f.s. In regular inspections to date no damage to the helical lining has been found.

Answering inquiries about the joints in the Pressure Aqueduct, one of their distinct features not found in the European "Bonna" pipe is the tight fit steel to steel, which keeps undue weight off the gasket. The spigot is made of a rolled special cross-section requiring no machining. These sections are cut to length, scarfed, rolled to circular shape and joined into a ring by welding. The bell is made

from 5/16" plate, 10" and 10½" wide. After the butt welds are chipped to remove excess metal, the rings are passed through flanged rolls to bend in the edge, which is later welded to the shell. Finally, they are stretched to exact size by an 8-segment hydraulic press, which expands the ring by about 3½" inches in circumference and produces true size and circular form, so that the rings will match within the specified tolerance of 1/32nd inch. To insure the true circular shape, the inner and outer forms, as well as the steel plate and reinforcing-rod assembly, are first placed in vertical position on a fixed, heavy, true, circular base ring; and a true, circular top ring which acts as a casting platfom is clamped in position over the spigot end.

To answer inquiries about quality of the concrete, the cement used was manufactured to the specifications developed by the late Thaddeus Merriman, and was found to be of excellent quality, giving a very tough, dense, high-strength and impervious concrete, which when properly mixed and cured included very little, if any, free lime and possessed a very high percentage of what Mr. Merriman called amorphous constituents, that is, particles which are not crystalline in structure and which are insoluble in water. A water-cement ratio of about 5 gallons per sack of cement was used at first and later increased to 5.8. At the beginning of the work, about 2 barrels of cement were used per cubic yard. As the work progressed, tests indicated that as little as 1.65 barrels could be used in warm weather without appreciable effect on the density. Even for the leaner mix, the compressive strength was in excess of 5,000 pounds per square inch.

Experience here indicates that we may expect some reduction in carrying capacity of concrete-lined pipes, tunnels, etc., due not to roughening of the concrete surface but to growths of organisms. These produce a thin, spongy slime, even at great distances from the intake works at the reservoir end. Such obstruction is not permanent, since it can be removed by heavy chlorination. We have not yet had an opportunity to make satisfactory full-capacity tests on the Hultman Aqueduct. On the Quabbin Aqueduct, 24.6 miles long and 127 square feet in cross-section, in which the original designs counted on an ultimate coefficient in the Kutter formula of n=0.013

and a probable early coefficient of n=0.012, the actual coefficient in 1943 was 0.01125.

Answering inquiries about the analyses of metropolitan water, recent records at Quabbin Reservoir near Quabbin Aqueduct Intake show the following in parts per million:

Color—12 to 17, averaging about 15
pH—6.6 to 6.9
Iron—0.05 to 0.10
Manganese—0
Dissolved Oxygen—8 to 11

It is easily possible to draw water from Quabbin Reservoir, even during the summer months, as cold as 55° F. The Department of Public Health reports analyses at service taps in Revere, at the extreme northeast end of the District, showing approximately the following:

Color—11
Hardness—16
Alkalinity—10
Iron—0.16

The supply is not filtered. Chlorine is added at several points, generally speaking where the supply is drawn from the open distributing reservoirs.

Answering questions about the use of the words "water wheel", these are used to describe any water-driven unit, including the reaction type.

The author appreciates the considerable interest shown and regrets that he was unable to present the paper in person and thus be in a position to answer inquiries more satisfactorily.

PROCEEDINGS

April 1946 Meeting

Hotel Statler, Boston, Mass.

Thursday, April 18, 1946

President Harold W. Griswold in the Chair.

Secretary Conroy announced the election of the following new members:

John J. Campobasso, Engineer, E. B. Badger & Sons Co., Boston, Mass.; Henry J. DiMestico, Superintendent, Water Department, Hanson, Mass.; William G. Dyer, Sanitary Engineer, War Department, 1st Service Command, Head of Water and Sanitation Section, Braintree, Mass.; Justin A. Holden, Superintendent, Water Department, Brattleboro, Vt.; Dr. J. R. Hurtado, Chief Engineer, Laboratory of National Institute of Sanitary Works, Caracas, Venezuela; Vail O. Leach, Municipal Manager and Superintendent of Water Department, Windsor, Vt.; Frederick F. Libby, Assistant Engineer and Superintendent, Water Department, Walpole, Mass.; Norman T. Marshall, Chief Clerk, Water Department, New Britain, Conn.; William L. Morse, Superintendent, Water Department, Hanover, Mass.; Clair N. Sawyer, Associate Professor, Sanitary Chemistry, M.I.T., Belmont, Mass.; Ariel A. Thomas, Instructor in Sanitary Engineering, M.I.T., Blackstone, Mass.; Harold C. White, Village Manager and Chief of Fire Department, Castleton, Vt.; Philip L. White, Town Manager, Brattleboro, Vt.; Richard M. Cleveland, N. E. District Manager, Worthington Pump & Machinery Corp., and Member of Water Board, Belmont, Mass.

President Griswold read a summary of a report by Warren J. Scott relative to a meeting which was held in New York on April 4, 1946, to discuss stream-pollution legislation. Mr. Scott represented the New England Water Works Association at the meeting. His report was summarized thus:

"After considerable discussion, the writer prepared a statement

which, after some editing, appeared to be satisfactory to all those present. This statement reads as follows:

" 'If federal legislation on water pollution control is to be enacted, our opinion with regard to such legislation is as follows:

" ' "The answer to water pollution control is the classification of the waters in the various states and between states, and the disposal and treatment of sewage and wastes, according to highest use of the waters involved, but this classification should be under the control of the states affected, with federal guidance and stimulation, rather than be placed entirely in the hands of central federal agencies clothed with mandatory powers. These principles are most nearly set forth in HR-4070 rather than other water pollution control bills now being considered in the Congress.' "

"It was decided to send this statement in a telegram to the chairman of the House Rivers and Harbors Committee, which is considering water pollution control bills, the telegram to be sent by Mr. Brush but with an accompanying note to the effect that this statement represented the opinion of the members gathered together from various associations, such as the American Society of Civil Engineers, the Conference of State Sanitary Engineers, the Federation of Sewage Works Associations, the American Water Works Association, the New England Water Works Association and the American Public Health Association.

"It was agreed that, after the Congressional committee reported out a bill, it might be desirable to call together the group again."

A talk on "Corrosiveness of Water to Metals—Introductory Remarks on Basic Principles" was given by Thomas R. Camp, Consulting Engineer, Boston, Mass.

A paper on "Electronic Principle of Water Purification" was read by Fred E. Stuart, President, Stuart-Brumley Corp., Baltimore, Md. The paper was supplemented by a practical demonstration.

A sound motion picture, "Eternally Yours," was shown by Arthur P. Grimm, Engineer, A. M. Byers Co., Boston, Mass.

May 1946 Meeting

Hotel Stratfield, Bridgeport, Conn.

Thursday, May 16, 1946

President Harold W. Griswold in the Chair.

Secretary Conroy announced the election of the following:

Members: Frank O'Connell, Water Commissioner, Holyoke, Mass.; Wilpas V. Kallio, Superintendent, Water Department, Rutland, Vt.; Russell J. Rice, with Metcalf & Eddy, Needham, Mass.; Nathan C. Avery, Plant Engineer, Southbury Training School, Southbury, Conn.; Joseph H. DeFoe, Engineer, Massachusetts Department of Public Health, Winthrop, Mass.; William F. Ablondi, Superintendent, Water Department, Seymour, Conn.

Associate Member: Moore & Co., Water Contractors and Engineers, Boston, Mass.

Upon motion of Warren J. Scott, seconded by Sidney S. Anthony, it was voted that the sum of $1,000 be added to the budget of the Association—$700 to be allocated to office expense, $150 to meetings and committees and the balance of $150 to a contingency fund.

Warren J. Scott and William W. Brush discussed a new bill, known as HR-6024, which had been approved by the Rivers and Harbors Committee of the House of Representatives.

The Honorable Jasper McLevy, Mayor of Bridgeport, extended his greetings to the members of the Association.

A paper "Construction of Norwalk Lake Dam" was read by John E. Riordan, Engineer, First Taxing District Water Department, Norwalk, Conn.

A paper "Comments on the Construction of the Saugatuck Dam" was read by Samuel P. Senior, President, Bridgeport Hydraulic Co., Bridgeport, Conn.

A paper "Experience with Break-point Chlorination in Connecticut" was read by Frederick O. A. Almquist, Principal Sanitary Engineer, Connecticut Department of Health.

MEMOIR OF WILLIAM R. CONARD

The water-works fraternity lost one of its well-known members through the sudden death of William R. Conard at his home in Burlington, N. J., on April 30, 1946, at the age of seventy-six.

Mr. Conard was an engineer and consultant and had spent the greater part of his life in connection with foundry and shop inspection work in the water-works field. During his professional career, which spanned nearly half a century, he had been engaged by a great number of communities, especially those located in the eastern part of the United States, including Boston and Springfield, Mass., and Albany, N. Y. During the first World War he was a consulting engineer on the building of the camp at Fort Dix, N. J., and the Bethlehem Steel Corporation's shell-loading plant at Mays Landing, N. J. Since July, 1945, Mr. Conard had been a member of the Council of Water Policy and Supply, New Jersey Department of Conservation.

He had been a member of the New England Water Works Association since November 12, 1902, and was an honorary member of the American Water Works Association. He was also a member of the American Society of Civil Engineers, the American Society of Mechanical Engineers, the American Society for Testing Materials and the Engineers' Club of Philadelphia.

In addition to leading a busy professional life, Mr. Conard was one of Burlington's most active civic, church and fraternal leaders. He had been postmaster of Burlington, a member of the City Council and president of the Board of Education. Recently he had been named secretary of the Burlington Post-War Planning Committee. At the time of his death he was president of the board of trustees of the Broad Street Methodist Church. He was the oldest past state councilor of the Jr. OUAM, a past master of the Burlington Lodge of Masons, a past commander of Knights Templar and a member of the Burlington Lodge of Odd Fellows.

Mr. Conard is survived by his widow, Mrs. Clarabelle Conard; two sons, Wilfred and Robert; two daughters, Miss Esther Conard and Mrs. Clarabelle Mitchell, and eleven grandchildren.

GEORGE C. HOUSER

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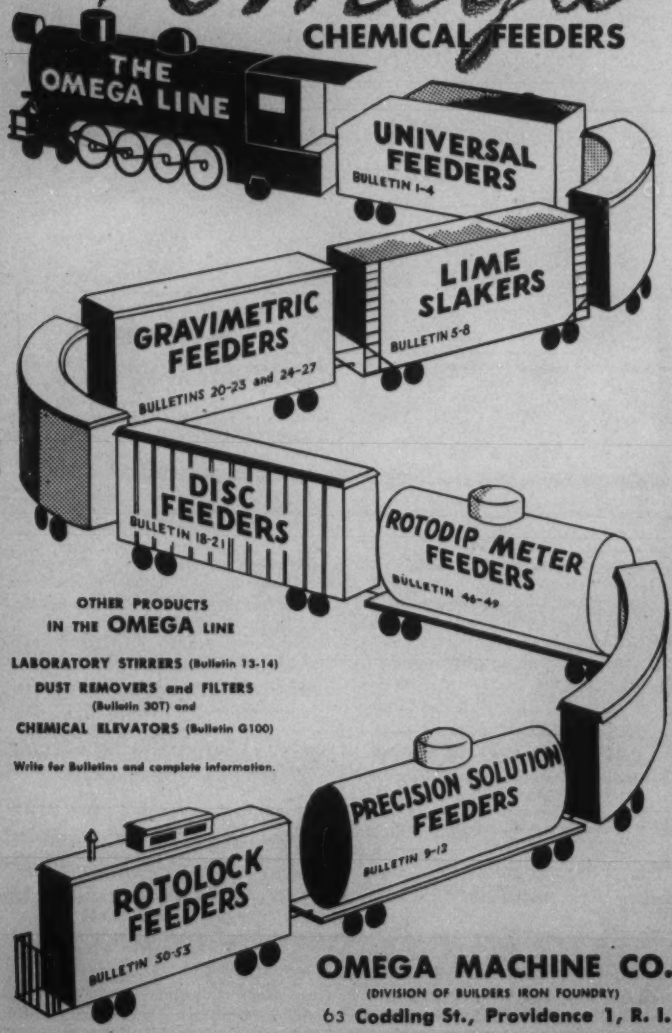

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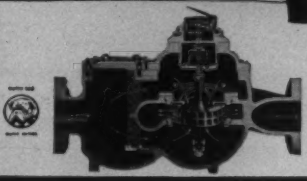

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THIS new Manual TELLS what is needed to set up a program of meter repair and testing . . . SHOWS how to lay out an efficient meter shop . . . PICTURES step-by-step repair instructions, presenting many hints that will save, and lead to greater meter accuracy over a longer period of years. It suggests practical ways to help you collect all possible revenue, prevent water waste, and distribute costs. The outline of its contents will show its value to large and small water utilities. Write for your copy today.

PLANNED METER REPAIR. Case histories from several cities with figures on decrease of unaccounted-for water, and increase in revenue. Suggested testing schedule for small meters.

PLANNING THE METER SHOP. Essential equipment needed. Typical work flow sheet and meter shop layouts. Typical shop photos. Novel "baker's tray" parts-storage arrangement.

TESTING MEANS DOLLARS SAVED. Actual test results on repaired meters. Importance of testing at minimum flow. Tests by GPM vs. orifice size. The "milk bottle" test.

METER RECORDS SHOULD BE ACCURATE. Reproduction of ideal record-card. Data that should be included.

REPAIR AND TESTING. Eight pages containing 20 step - by - step - detail how-to-do-it photos showing high-spots in meter repair. Tools needed in a small shop. "Upside-down" storage. Further "suggestions" that will interest the repair man.

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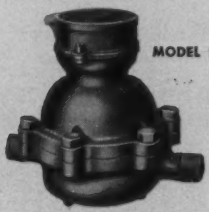

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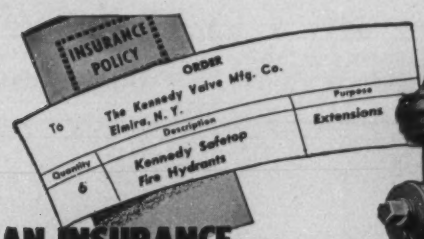

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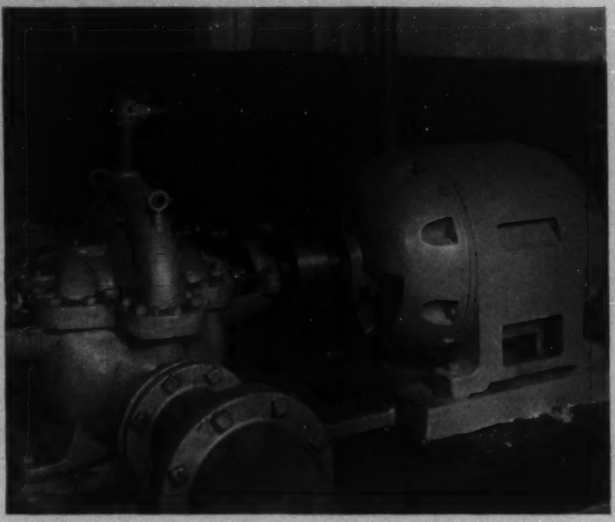

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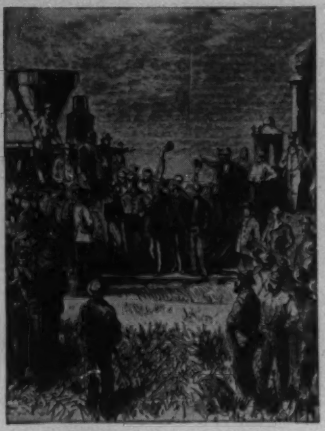

Famous Facts

"The Wedding of the Rails" — the driving of the golden and silver spikes to symbolize the joining of the Central Pacific and the Union Pacific lines — at Promontory Point, Utah, May 10, 1869, marked the completion of the first transcontinental railroad. Four years before, the Union Pacific had begun to lay its 1,085.8 miles of track westward from Omaha, Nebraska. At the same time, the Central Pacific was coming eastward with its 690 miles.

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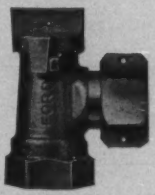

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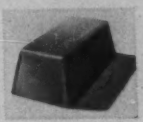

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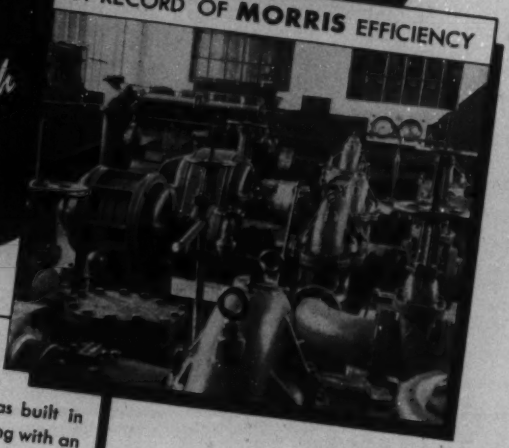

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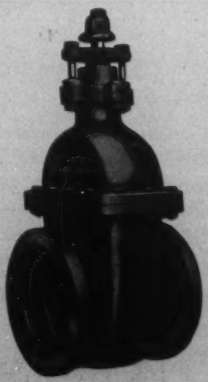

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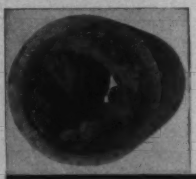

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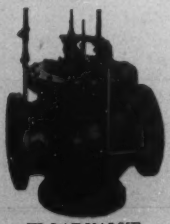

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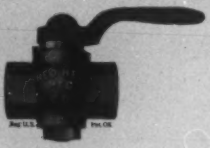

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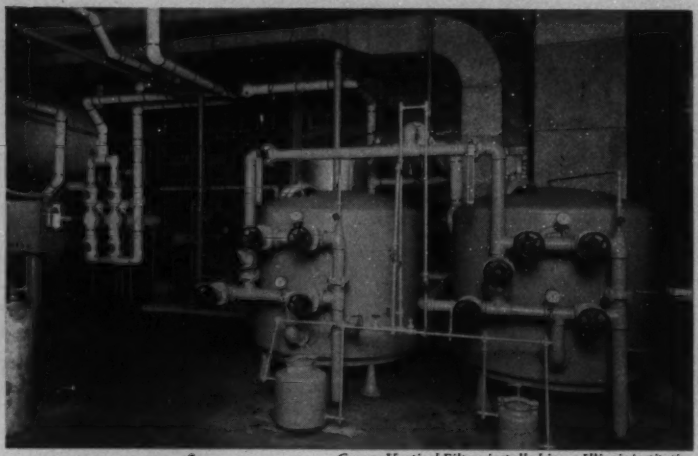

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